SEWAGE DISPOSAL

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PREFACE

WHEN this book was begun, neither H. P. Raikes' "Design, Construction and Maintenance of Sewage Disposal Works" nor H. T. Calvert's translation of Dunbar's "Principles of Sewage Treatment" had been published. Even after these excellent books appeared, however, it still seemed to us that there remained room for a general survey of the problem from the various viewpoints of the chemist, the sanitary biologist and the engineer, and with particular reference to the conditions of American practice. It has been our aim to discuss somewhat fully the fundamental principles of chemistry and bacteriology which are involved and yet to include also the more important aspects of the engineering works designed to carry them into operation. It is hoped that the book may be useful to the student of sanitary engineering who aims to fit himself for the construction of sewage disposal works, to the engineer who after working in other lines is drawn into this growing field, and to the chemist, the bacteriologist and the public health official concerned in the operation of disposal works after they are built.

The thanks of the authors are due to a long list of engineers and others who have furnished unpublished data for various sections of the book. In particular, grateful mention must be made of Mr. J. D. Watson, of Birmingham, England; Mr. G. J. Fowler, of Manchester, England; Mr. W. J. Dibdin, of London, England; Mr. H. P. Eddy, Mr. Leonard Metcalf and Mr. F. A. Barbour, of Boston; Mr. G. E. Bolling, of Brockton, Mass.; Mr. W. M. Brown, of the Metropolitan Sewerage Commission of Massachusetts; Mr. A. L. Fales, supervising chemist Worcester Sewage Works; Mr. J. W. Bugbee, supervising chemist Providence Sewage Works; Mr. Charles Saville, of the Massachusetts State Board of Health, Mr. F. W. Jones, Worcester Polytechnic Institute, and Mr. Paul Hansen, of the Ohio State Board of Health.

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SEWAGE DISPOSAL

INTRODUCTION

THE SANITARY DEMAND FOR SEWERAGE AND SEWAGE DISPOSAL

The Wastes of Human Life. The human body is a biological machine which requires food for fuel and which produces in its operation a considerable amount of waste material. Like the ash from a furnace, this waste consists partly of mineral matter and partly of incompletely oxidized fuel. The excretions of the kidneys, representing the end products of cell metabolism, still contain a large amount of organic matter in the form of urea; the discharges from the alimentary tract consist largely of undigested, or partially digested, foods which have not been absorbed by the body. All these substances undergo further change after they are excreted from the body, breaking down into simpler or more stable compounds, and during this change compounds are formed which are characterized by the penetrating noxious odors of putrefaction.

Besides the organic waste materials from digestion and excretion, the excreta contain a host of microscopic living organisms, and this fauna and flora is of even greater practical importance than the lifeless substratum upon which it subsists. The surfaces of the human body, within and without, are parasitized by micro-organisms, which find their most favorable conditions for multiplication in the digestive tract. MacNeal, Latzer and Kerr (1909) report an average of 33 million million bacteria per day in the feces of normal adult men;* and Proto-

* Complete references to all literature cited will be found in the bibliography at the end. References in the text include the name of the authority (the initials in the case of the British commissions) and the date of publication, with a distinguishing letter in case more than one volume appeared in a single year. This serves simply to identify the article or book, the full title of which is given in the bibliography.

zoa, though not so ubiquitous, are by no means rare. Most of these microscopic parasites are harmless, putrefactive forms; but in the excreta from a patient suffering with typhoid fever, or dysentery, or cholera, or any other intestinal malady, the specific germ of the disease may at any time be present.

The discharges from the alimentary canal and the kidneys do not of course exhaust the catalogue of human wastes. The washings of the outer skin, and the wash water from cooking and house-cleaning, are included in the wastes from the household. In more closely settled communities, street washings and the wastes from industrial establishments are also added. With the exception of factory wastes and street washings, which are of various composition and require special treatment, the excretions of the body may be taken as a general type. The important constituents in every case are the intermediate products of organic decomposition plus the living micro-organisms, which may at any time include specific pathogenic forms.

Primitive Methods of Dealing with Excreta. Under primitive conditions excretal matter ultimately finds its way to the water or the earth. Direct discharge into watercourses is eminently satisfactory to the persons immediately concerned, if the flow be sufficient to prevent local accumulations. The effect of this procedure upon individuals and communities on the watershed below may, however, be serious. This is an aspect of the larger problem of sewage disposal and stream pollution which will be discussed more fully further on; as far as the polluting individual goes, direct discharge into water is an efficient method of disposal. The earth, too, is able to assimilate decomposing organic matter with success, as demonstrated yearly by the manuring of the fields. Mixed with a sufficient quantity of earth, and with reasonably free access of air, excreta are quickly disintegrated and oxidized to stable and innocuous forms.

The difficulty with the method of earth disposal lies in its application. Its success demands prompt and complete mixture with clean dry earth; and this is rarely attained. The

conditions which actually exist are various. The most primitive houses are provided with privy vaults for the excreta, but discharge sink and other wastes directly on the surface of the ground, with or without the medium of a drain. Combinations of privy vaults for excreta with cesspools for sink drainage lead up to a better class of houses provided with cesspool connections for all wastes.

The privy vault is certainly the most objectionable of these contrivances. It stores up quantities of human excreta in a slowly decomposing condition. It is generally loosely built, and the material which it contains is more or less freely exposed to the air and to the distributing agency of insects and higher animals. It is subject to overflow and surface discharge at times of heavy rain; and the material which it contains must generally be removed and otherwise disposed of at intervals, the process of handling causing fresh nuisance and menace to health. A well-constructed cesspool is free from many of these objections. The material in it is, or should be, closed in, so that air-borne odors and the access of insects are prevented. The liquid contents of the cesspool must go somewhere, however. If the surrounding soil be of a suitable sand, the liquid may so filter through it as to be efficiently purified. Such a cesspool may operate for years without filling up and without causing pollution of water or earth. Where, however, a rocky or clayey soil is traversed by fissures, a leaching cesspool may constitute a serious menace to well waters, even at considerable distances. A tight-walled cesspool, on the other hand, resembles a privy vault in the danger that it may overflow and in the certainty that it must be emptied and the material in it removed and carted away.

The Dangers from Accumulations of Excretal Material. Accumulations of excretal matter are not merely objectionable on account of the unpleasant odors due to their decomposition. The nuisance from an individual cesspool or sink-drain is seldom sufficient to be obnoxious, except in the immediate vicinity. The real danger lies in the disease germs which

may be present. Typhoid fever and dysentery are unfortunately still common diseases; and in the discharge of persons suffering from these, or other, intestinal disorders the specific parasites may be present in great numbers. Even with patients patently suffering from such maladies it is rare that proper precautions are taken for disinfection. Unrecognized cases of a light nature may spread the most virulent germs; and occasionally "typhoid carriers" are found, infected with pathogenic organisms, though not themselves suffering from the disease. Wherever excreta are present the germs of intestinal disease are potentially present also.

If excretal matter carrying disease germs is kept exposed in privy vaults, or is discharged from overflowing cesspools, the transfer of the infection to susceptible human beings is one of the most probable of events. Flies, which impartially affect privies and larders, offer perhaps the most convenient mechanism for transferring fecal matter to the mouth. Rats and chickens and other animals play their part. Children may easily acquire more direct infection. The carelessness which is encouraged by the uncleanly environment of the privy vault increases the danger to all who use it; and, finally, the emptying of privy or cesspool and the transportation of the contained material to its point of final disposal, spreads infection in a hundred ways.

The statistics of typhoid fever in the Spanish-American war offered striking proof, if any were needed, of the relation between that disease and the care of excreta. The chief cause of the scandalous prevalence of typhoid (which affected one-fifth of all the troops in the national encampments of the United States) was camp pollution. The official report of Surgeons Reed, Vaughan and Shakespeare (1904) concludes, "It may be stated in a general way that the number of cases of typhoid fever in the different camps varied with the methods of disposing of the excretions." In the Seventh Army Corps, for example, the First Division had a water-carriage system of disposal and developed 173 cases of typhoid fever per regiment, on

the average. In the Third Division regulation pits were used, in which the excreta were supposed to be promptly covered with earth. This method is fairly satisfactory, though inferior to water carriage. There were 185 cases per regiment in this Division. The Second Division used the tub system of disposal, by which "infected fecal matter was scattered all through the camp," and had an average of 299 cases per regiment.

Sanitary Dry Closets. Where a water supply is not available it is obviously impossible to secure the immediate removal of excreta, and the only thing to do is to minimize as far as possible the dangers which have been discussed in the preceding section. For this purpose various methods of dry disposal have been devised, dating back at least to the excellent sanitary regulations in the twenty-third chapter of Deuteronomy.

In a temporary camp the pit system of disposal is probably the best available. Here, the excreta are received in an ample trench and at once covered with a few spadefuls of dry earth or ashes. At frequent intervals the trenches should be filled in and new ones excavated. This is of course merely a modified privy vault with provision for covering in the excreta. In connection with permanent dwelling houses the pail system, (*fosses mobiles* of the French), should be installed. In this system tight pails or tubs are placed under the closet seat to receive the excreta and are removed and emptied at frequent intervals. A supply of ashes and earth should be at hand for absorbing liquids and exercising a deodorant action.

In many German cities the pail system has attained a considerable development and the excreta from large communities are handled in this way. For final disposal, the material collected from an isolated house may be carried, tightly covered, to some point at a distance from the dwelling and dug into the ground. Where larger settlements use the pail method, the collected excreta may be used in their crude condition for manure or they may be worked into artificial fertilizer, or they may be burned in a cremator designed for the purpose.

The following sanitary essentials for a dry-closet disposal sys-

tem are enumerated by Blasius (1894), in connection with an excellent discussion of German systems of this type.

1. Pails of adequate capacity and complete impermeability.
2. Tight connection between pails and closets.
3. Constant ventilation of closet rooms and closets.
4. Regular and frequent removal of pails.
5. Hermetical closing of pails in transport.
6. A pail chamber under the closet, protected from frost and from the heat of the sun and provided with an impermeable floor. This chamber should open from outside the house.
7. Complete cleansing and disinfection of the empty pails before they are replaced.

The Water-Carriage System. The pail system of removal is a makeshift at best. It may operate well under rigid supervision, but it involves too much machinery and too much handling for constant sanitary efficiency. The ideal method of removing excreta is by the water-carriage system. With water supply and sewerage and proper plumbing the excreta are at once washed away into a system of closed pipes and removed promptly and completely from the vicinity of the dwelling. Wherever water is available and the construction of sewers is economically possible, no other system of handling fecal matter should be tolerated by sanitary authorities.

The recognition of these facts is a comparatively recent event and the use of the water-carriage system for removing excreta is essentially a modern one. The Cloaca Maxima and the other so-called sewers of antiquity were rather drains than sewers, and their function was to lower the ground-water level and to carry surface water rather than to remove excreta. Until 1815 the discharge of any waste but kitchen slops into the drains of London was prohibited by law, and the same regulation persisted in Paris up to 1880. Sewerage and sewage disposal proper really date from the epoch-making report of the health of towns commission of Great Britain in 1844, which revealed the accumulation of such an astonishing amount of decomposing organic matter and filth of all kinds in the cities that it aroused

British sanitarians to a strong movement for the amelioration of these conditions. Public and private cleanliness was taught and practiced as never before. The midden system and the pail system rapidly gave way to the water-carriage system. Whereas in 1815 the sewers of London were simply drains to carry off the storm water, in 1847, only three years after the report of the health of towns commission, it was made obligatory to discharge all sewage into those drains.

In other countries the example set in England was more or less promptly followed. In the United States numerous drainage systems existed, — one in Boston, for example, dating from the seventeenth century; but the first comprehensive sewerage project was designed by E. S. Chesbrough for the city of Chicago in 1855. On the continent of Europe a sewer system was constructed at Hamburg after the great fire of 1842, by Lindley, an English engineer. Berlin began her sewerage in 1860 and other German systems quickly followed.

Efficient Sewerage Systems and the Death Rate. The new method of dealing with excreta quickly justified itself by its results. A marked decline in the death rate, and particularly in the typhoid death rate, has followed the introduction of sewerage systems. In many cases simultaneous improvements in water supply complicate the results; but in a number of instances it seems clear that the removal of excreta was the main force at work. Thus Pettenkofer (1874) shows that at Munich the typhoid death rate was 242 per 100,000 between 1852 and 1859. Improvements in privy vaults and the construction of sewers began between 1856 and 1859. From 1860 to 1867 the typhoid death rate fell to 166. New water supplies and other reforms have since reduced the death rate to a very much lower point; but this first diminution of one-third was primarily the result of sewerage.

In Berlin, Weyl (1893) records similar phenomena. The introduction of a public water supply in 1856 produced a rapid decrease in typhoid fever; but the opening of the first considerable system of sewers in 1876 caused the curve of typhoid to

take a much steeper fall than its previous course would have indicated. The curve for the ratio of typhoid deaths to total deaths in Berlin, during this period, is plotted in Fig. 1.

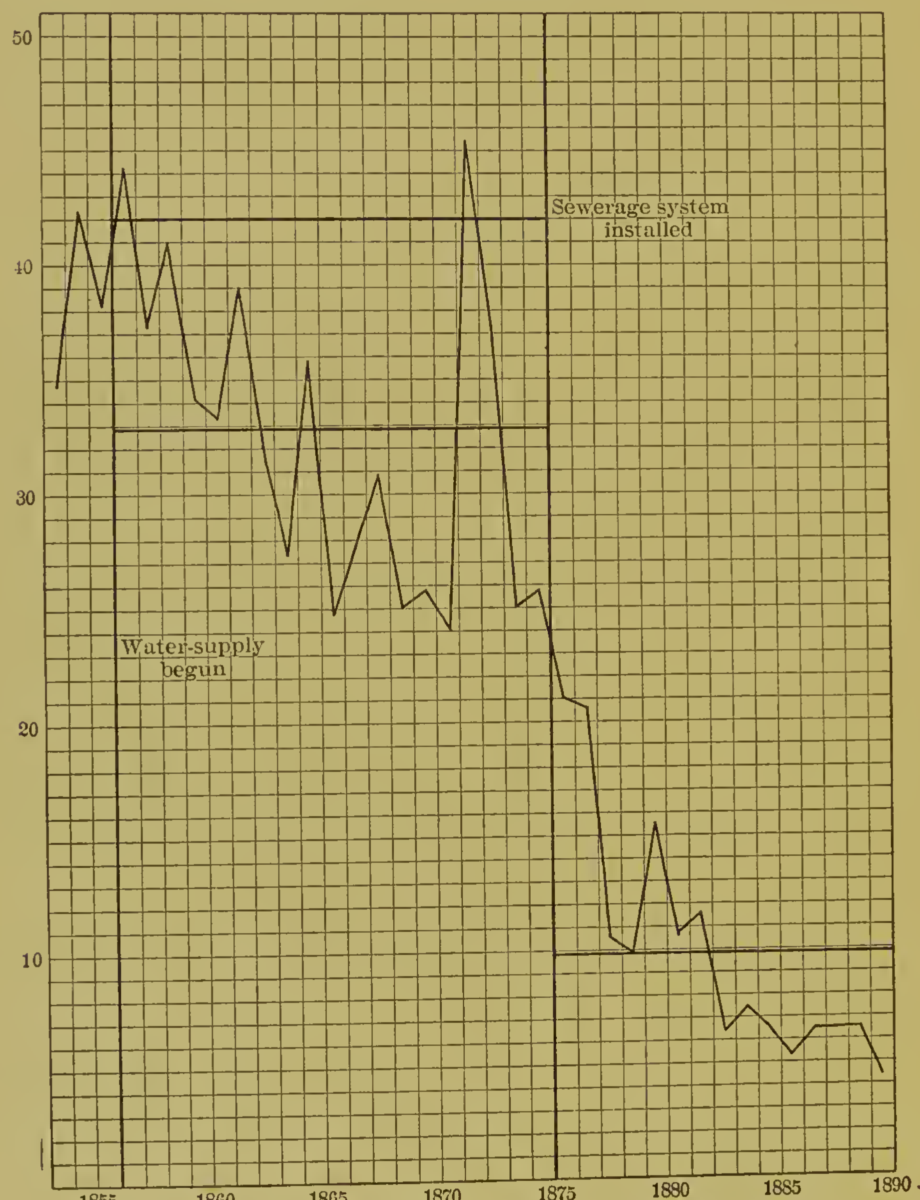


Fig. 1.—Effect of Water Supply and Sewerage on the Typhoid Death-rate of Berlin (Blasius, 1894).

Parkes and Kenwood (1907) cite the case of Nottingham, England, in illustration of this point. In that city, "where middens, pails, and water closets are in use in different parts of the town, Dr. Boobbyer has shown that the greatest prevalence of enteric fever is to be found in the houses with middens, and the least in the water-closeted houses, those with pails occupying an intermediate position. In 1902 there were twice as many cases of enteric fever proportionally in 'pail' houses as in 'w. c.' houses, and 14 times as many cases in 'midden' houses as in 'w. c.' houses."

The Sanitary Significance of Drainage. There is another aspect of the sewerage problem which deserves some consideration,— the sanitary importance of drainage, pure and simple, as distinct from sewerage. A sewerage system not only carries away excreta; it furnishes also, if desired, an opportunity for the removal of surface water; and it may be so constructed, or so supplemented by subdrains, as to effect a permanent lowering of the ground-water level. The removal of accumulations of surface water is obviously of much sanitary importance, since stagnant water offers an opportunity for the *Anopheles* mosquito to breed and thus promotes the spread of malaria. The lowering of subsoil water is also a sanitary desideratum under certain conditions. In the classic investigations of Buchanan in England (Buchanan, 1867) it appeared that where sewerage systems led to an appreciable drying of the soil a marked diminution of tuberculosis followed, whereas in towns where no such drying took place the tuberculosis death-rate was stationary. In this case the direct effect of a drier air on vital resistance was no doubt the efficient cause.

The New Problem of Sewage Disposal. Water carriage is clearly the ideal method of sewerage for the individual householder. It removes excreta and all other liquid wastes promptly and completely from the region of habitation; it prevents contamination of water and earth; and it offers an opportunity for the drying of surface soil and subsoil. The problem of ultimate disposal is, however, merely shifted from the individual to the

community. The unsanitary condition surrounding the dwelling is relieved, but at some point on the outskirts of the city the concentrated filth from the entire population must be delivered and must be taken care of by municipal authorities, and the problem is rendered all the more difficult by the large amount of water which carries this filth through the sewers to the point of discharge. The proper disposal of the combined waste of the community so that it shall not cause offensive or dangerous conditions, is the problem of sewage disposal. The magnitude of this problem may be realized by the simple statement that from the sewers of the Metropolitan District of Boston 186 million gallons of sewage containing 19 tons of nitrogen are discharged each day into Boston Harbor; and the extent to which this problem is still unsolved is indicated by a statement of Fuller's (1905 *a*) that in 1905 only 28,000,000 of the inhabitants of the United States were served by sewerage systems, and only 1,100,000 were served by sewerage systems connected with sewage purification plants. Since that time certain large cities, as, for instance, Columbus, have constructed sewage disposal works, but Fuller's figures still remain approximately true; and it is with the hope that the methods by which sewage can be prevented from causing a nuisance, and from being a source of potential danger, may be brought to the attention of students of sanitary science, that the following pages have been written.

CHAPTER I

COMPOSITION OF SEWAGE

General Characteristics of Sewage. Sewage may be defined as the water supply of a town or city after it has been used. It is water polluted by the solid and liquid excreta of the population, household waste, the refuse of various industries, and street washings. It is water containing mineral, vegetable and animal matter in suspension and in solution.

Besides the mineral and organic matter, sewage always contains vast numbers of living organisms, — bacteria, — twenty-five to fifty million in a liquid ounce; and though by far the greater number of these bacteria are harmless to man and essential agents in the removal of the organic matter in sewage, there are also present bacteria which are far from harmless, the so-called pathogenic forms; and under certain conditions the removal of pathogenic bacteria, as well as the destruction of the organic matter in sewage, must be considered.

In appearance, sewage, as seen running in the sewers, resembles the dirty water in a washbowl, containing, however, floating on its surface, fecal matter, bits of paper, matches, fruit skins, vegetables, etc. It has only a slight odor and is comparatively inoffensive. If placed in a glass cylinder after being passed through a coarse strainer, and allowed to stand for twenty-four hours, a slimy, greasy matter settles out, but the liquid remains turbid, and offensive odors are given off, caused by the putrefaction of the animal and vegetable matter which it contains. Finally the liquid becomes clear, the odor disappears, and the sediment which remains is no longer in a state of putrefaction, having been reduced to a condition similar to that of the humus of the soil.

The change is typical of what always takes place when animal and vegetable matter is exposed to the action of the air, and to bring about this change quickly and inoffensively is the object of all methods of sewage treatment and disposal.

Successful treatment of the sewage of a given community depends largely on the adaptation of the method used to the work required, and this depends not only on the volume of sewage to be treated, but also on the amount and nature of the solid matter. How much of the solid matter is mineral, how much vegetable and animal; how much is in solution in the water, how much in suspension; what decomposition the nitrogenous matter has undergone, how much is still wholly or partially undecomposed; what is the general nature of the undecomposed organic matter, and what substances will be formed during its decomposition: are all questions that must be taken into account in the study of the sewage from a given community.

As a general statement it may be said that the sewage from an American residential town contains in one million parts of sewage from 200 to 800 parts of solid matter (two-tenths to eight-tenths of a gram per liter), or less than one-tenth of one per cent; that about one-half of this solid matter is mineral, the other half vegetable or animal; and that about 75 per cent of the mineral matter and 60 per cent of the vegetable and animal matter is in solution. This can be expressed in tabular form as follows:

Solid matter, 200–800 parts per million.	Mineral, 50%, inoffensive..........	In solution, 75%
		In suspension, 25%
	Vegetable and animal, 50%, offensive	In solution, 60%
		In suspension, 40%

Variations in Composition of Sewage. There is a great variation in the amount and character of the solid matter in the sewage flowing in the sewers of different towns and cities. These variations depend on the quantity of water used, the leakage of ground water into the sewers, the quantity and

kind of manufacturing waste discharged, and various other factors.

The mineral matter present consists chiefly of sand, clay, iron and aluminium oxides, and the chlorides, carbonates, sulphates and phosphates of the alkalies and alkaline earths. The portion of this mineral matter that is in solution is generally of little consequence in relation to sewage treatment, though conclusions as to the concentration of domestic sewage may be drawn from the amount of chlorides present, since urine, excreta and kitchen waste all contain sodium chloride. As to the insoluble mineral matter, largely sand and clay, which the sewage may contain in large amounts when street washings are present, provision must be made for its removal in all methods of sewage treatment.

The character of the organic matter in sewage is of much greater importance, as the nuisances that arise from sewage are due to the breaking up of the vegetable and the animal substances it contains. The organic substances in sewage can be divided into compounds containing nitrogen and compounds free from nitrogen. The principal nitrogen compounds are urea, proteids, amines and amido acids; the non-nitrogenous compounds are carbohydrates (including cellulose), fats and soap.

It is not as yet possible by any method of analysis to tell the exact amount of either the nitrogenous or the non-nitrogenous matter present; and in determining the character and concentration of a given sewage, we are obliged to be content with knowing the amount of total and suspended matter; the amount of each that is volatile at a low red heat, often called the organic matter; the amount of fat or grease; the total amount of chlorides; the total amount of nitrogen in the organic matter present, called organic nitrogen; the amount of nitrogenous matter that is easily decomposed, as shown by the nitrogen that will be set free as ammonia when the sewage is heated with an alkaline solution of potassium permanganate, called albuminoid ammonia; the amount of ammonia that has been formed by the

natural decomposition of the nitrogenous organic matter, called free or saline ammonia; the amount of nitrogen united to oxygen in the form of nitrites and nitrates, which, taken in connection with the free ammonia, indicates the amount of change that the nitrogenous matter in the sewage has undergone; and, finally, the amount of oxygen that will be given up from an acid solution of potassium permanganate when sewage is treated with this compound, which indicates the amount of oxygen that is required to change the organic matter into water, carbon dioxide, sulphur trioxide and nitric acid.

The determination of the above data constitutes what is known as a sewage analysis, and though the knowledge that can thus be obtained is far from being all that could be desired, it yields valuable information, not only in regard to the amount of total solid matter present in the sewage, but also in regard to the amount of organic matter and the character of the nitrogenous substances present.

It must be clearly understood, however, that an analysis of a casual sample of any given sewage will give no correct idea as to the character of that sewage, as the sewage of any community may vary greatly from hour to hour; and it is only from analyses of a large number of samples collected at frequent intervals, or analyses of one or more series of half-hour samples taken during several consecutive days, that any reliable information can be obtained.

Of late years many such analyses of sewage have been made, a few of which are quoted in the following tables:

TABLE I

ANALYSES OF SEWAGE OF SMALL RURAL COMMUNITIES IN MASSACHUSETTS, CONTAINING LITTLE TRADE WASTE. FLOW LESS THAN 150,000 GALLONS IN 24 HOURS.

		Parts per Million.			
		Andover.	Stockbridge.	Leicester.	Average.
Inhabitants contributing sewage		3,600	800	500	.....
Total dry-weather flow		125,000	75,000	30,000	.....
Flow per person connected with sewer, U. S. G.		35	94	60	63
Total solids	Total	466.5	235.6	421.7	374.6
	Fixed	222.6	147.6	201.2	190.5
	Volatile	243.9	88.0	220.5	184.1
Suspended solids	Total	103.3	40.5	140.1	94.6
	Fixed	17.1	9.6	22.8	16.5
	Volatile	86.2	30.9	117.3	78.1
Dissolved solids	Total	363.2	195.1	281.6	279.9
	Fixed	205.5	138.0	178.4	173.9
	Volatile	157.7	57.1	103.2	106.0
Nitrogen as	Free NH_3	39.7	9.8	22.0	23.8
	Alb. NH_3	5.6	1.6	4.1	3.8
Oxygen consumed in 5 minutes		49.0	15.2	50.8	38.3
Chlorine		70.0	12.8	54.7	45.8

AUTHORITIES (Tables I–III)

Tables I–II: Mass., 1904, pp. 305–455. Boston: October, 1905, to June, 1907. Winslow and Phelps, 1907, p. 410. Lawrence: Day sewage, 6 A.M. to 5 P.M. Average, 1908. Clark and Gage, 1909, pp. 15 and 17. Columbus: Johnson, 1905, p. 28. Providence: Bugbee. Personal communication, July 12, 1909. Average of four weekly analyses of hourly samples made July 25 to August 22, 1909. Worcester: Fales. Personal communication, Aug. 28, 1909. Average of hourly samples, taken day and night for three years, 1905–1908.

TABLE II

ANALYSES OF SEWAGE OF SMALL CITIES IN MASSACHUSETTS, CONTAINING SOME TRADE WASTE. FLOW BETWEEN 250,000 AND 1,250,000 GALLONS IN 24 HOURS.

		Parts per Million.						
		Brockton.	Framingham.	Gardner.		Marlboro.	Westboro.	Average.
				Gardner.	Templeton.			
Inhabitants contributing sewage		25,000	7,500	3,500	4,500	10,000	3,000	.....
Total dry-weather flow		878,000	652,000	302,000	250,000	1,100,000	282,000	.....
Flow per person connected with sewer, U. S. G.		35	87	86	55	110	94	77.8
Total solids	Total	818.8	587.7	383.7	430.6	448.2	370.1	506.5
	Fixed	393.6	288.9	154.4	160.9	229.3	157.1	230.7
	Volatile	425.2	298.8	229.3	269.7	218.9	213.0	275.8
Suspended solids	Total	194.6	212.4	154.0	201.8	137.5	183.3	180.6
	Fixed	21.2	50.2	23.6	24.2	21.8	46.4	31.2
	Volatile	173.4	162.2	130.4	177.6	115.7	136.9	149.4
Dissolved solids	Total	624.2	375.3	229.7	228.8	310.7	186.8	325.9
	Fixed	372.4	238.7	130.8	136.7	207.5	110.7	199.5
	Volatile	251.8	136.6	98.9	92.1	103.2	76.1	126.4
Nitrogen as	Free NH_3	42.9	26.1	20.2	27.3	26.0	13.8	26.0
	Alb. NH_3	7.9	6.5	4.9	6.6	4.4	4.5	5.6
Oxygen consumed in 5 minutes		162.7	47.3	49.2	60.3	44.4	35.5	66.6
Chlorine		131.8	69.9	33.8	43.8	59.0	23.7	60.3

TABLE III

ANALYSES OF SEWAGE FROM SOME OF THE LARGER CITIES IN THE UNITED STATES.

(From published reports and personal letters. See Footnote p. 5.)

		Parts per Million.				
		Boston.	Lawrence.	Columbus.	Providence.	Worcester.
Number of inhabitants connected with sewers		350,000	.....	100,000	189,000	127,100
Total dry-weather flow, U. S. G.		85,000,000	.....	9,100,000	18,663,000	15,000,000
Flow per capita		245	.....	91	99	118.2
Total solids	Total	.....	717	1026	1715	873.5
	Fixed	.....	430	836	994.5	439.2
	Volatile	.....	287	190	720.5	434.3
Suspended solids	Total	135	149	215	397	255.8
	Fixed	44	36	134	53.5	78.0
	Volatile	91	113	81	343.5	177.8
Dissolved solids	Total	.....	568	811	1318.0	617.7
	Fixed	.....	394	702	941.0	361.2
	Volatile	.....	174	109	377.0	256.5
Nitrogen as	Free NH_3	13.9	41.7	11.5	18.4	17.13
	Total alb. NH_3	.....	6.6	....	8.86	7.33
	Dis. alb. NH_3	.....	3.4	....	4.28	3.34
	Sus. alb. NH_3	.....	3.2	....	4.58	3.99
	Total organic N	9.1	.....	9.7	....	....
	Dis. organic N	5.8	.....	3.6	....	....
Oxygen consumed	Total	56	55.7	56	93.6	120.7
	Dissolved	43	.....	28	46.2	....
Chlorine		.....	143.0	67	(540.5) *	114.0

* Contains sea water.

TABLE IV

ANALYSES OF SEWAGE OF ENGLISH CITIES CONTAINING TRADE WASTE.

(From published reports and personal letters.)

		Parts per Million.						
		Birmingham.	Bradford.	Leeds.	Leicester.	Manchester.	Sheffield.	Average.
Number of inhabitants connected with sewers		900,000	240,000	454,450	235,000	575,000	400,000	.....
Total dry-weather flow, U. S. G.		32,544,000	15,600,000	20,648,400	11,551,244	30,360,000	19,200,000	.....
Flow per capita, U. S. G.		36	65	45.4	47.4	52.8	48	49.1
Total solids	Total	1947	2650	1843	1805	1235	....	1896
	Fixed	....	1774	....	....	592	....	.....
	Volatile	....	876	....	....	643	....	.....
Suspended solids	Total	718	840	775	680.3	327	417.5	668
	Fixed	....	264	....	....	163	170.7	.....
	Volatile	....	576	....	....	164	246.8	.....
Dissolved solids	Total	1229	1810	1069	1,124.7	908	....	1228
	Fixed	....	1510	....	869.2	429	....	.....
	Volatile	....	300	....	255.5	479	....	.....
Nitrogen as	Free NH_3	41.6	47.8	22.8	45.064	29.3	32.18	36.46
	Total alb. NH_3	16.2	32.8	7.88	15.6	7.0	9.02	14.75
Oxygen consumed in 4 hrs. at 80° F.	Total	259.2	202.3	99.0	125.22	113.6	79.6	146.5
	Dissolved	138.9	....	....	....	....	....	.....
Chlorine		205	149	165	127.2	200	124.3	161.8

Birmingham: Watson. Birmingham Sewage Disposal Works, Proc. Inst. of Civil Engineers, 1910. Average for 1901–1905.
Bradford: Garfield. Personal communication, August, 1909. General average.
Leeds: Hart. Personal communication, July, 1909. Average of 23 analyses.
Leicester: Mawbey. Personal communication, September, 1909. Average of 50 analyses, loss on ignition, 25.
Manchester: Fowler. Personal communication, August, 1909. Average for 1905.
Sheffield: Wike. Personal communication, August, 1909. Average of 130 analyses.

The above tables give a general idea of the nature of sewage, and show that the amount of total solid matter, organic matter and nitrogenous matter, varies within wide limits in the sewage of different communities; that the sewage of a large city is very different from the sewage of a rural community; and that the sewage of English cities (the same is true of Continental cities) contains more solid matter in a given volume, or, as usually stated, is more concentrated, than the sewage of American cities.

Calculation of Sewage Constituents in Grams per Capita. Analyses of sewage like those given are also used to obtain tables known as Grams per Capita tables, by reducing the results of analyses and the gaugings of the sewage flow to a basis of amount in weight of the various constituents per capita per day. For this purpose the following formula can be used:

Grams per capita per day $= A \times C \times 0.001 \times 3.785$:

when A = parts per million; C = gallons per capita per day; 3.785 = number of liters in a U. S. gallon.

Such tables may be of some assistance in forming a judgment of the relation between the number of people served by a sewer system and the probable quality of the sewage. It is not safe, however, to assume that such a calculation, based upon the data ordinarily available, even when taken from several places and averaged, will apply with reasonable accuracy to other cities. This is clearly demonstrated by the wide variations in Tables V and VI, compiled from the data given in Tables I and II.

These towns are all sewered on the separate system and the sewage is what would be classed as domestic sewage, although there are considerable amounts of industrial waste in the sewage of Brockton, and small amounts in the sewage of some of the other towns.

The variations are no doubt partly due to errors in sampling the sewage, measuring the flow, and estimating the population connected with the sewer systems. The quantity and character of ground water has a marked influence upon such data, as does also the surface water from streets and areas,

which not infrequently finds its way into "separate" sewers, — regulations to the contrary notwithstanding. Such connections, made from time to time, are seldom recorded and are soon forgotten, though they may have a very marked influence upon the composition of the sewage.

Manufactural wastes are extremely variable in quality, as well as in proportion to the quantity of domestic sewage. In many cities the quantities of the various ingredients usually determined may come as largely from industrial works as from the population. It is obvious, therefore, that there is no logical relation between the quantities of impurities in sewage from an industrial city and the population served by the sewer system. This is well shown in Table VII, which is a table of Grams per Capita for some of the principal cities of England calculated from Table IV.

TABLE V

QUANTITIES OF PRINCIPAL CONSTITUENTS IN GRAMS PER CAPITA DAILY. SEWAGE OF SMALL RURAL COMMUNITIES IN MASSACHUSETTS, CONTAINING LITTLE TRADE WASTE.

(Calculated from table on page 5.)

		Grams per Capita.			
		Andover.	Stock-bridge.	Leicester.	Average.
Flow per person connected with sewer, U. S. G.		35	94	60	63
Total solids	Total	62	84	96	89
	Fixed	30	53	46	45
	Volatile	32	31	50	44
Suspended solids.	Total	14	14	32	23
	Fixed	2	3	5	4
	Volatile	12	11	27	19
Dissolved solids	Total	48	70	64	67
	Fixed	27	50	41	42
	Volatile	21	20	23	25
Nitrogen as	Free NH_3	5.3	3.5	5.0	5.68
	Alb. NH_3	0.7	0.6	0.9	0.91
Oxygen consumed in 5 minutes		6.5	5.4	11.5	9.14
Chlorine		9.0	4.6	12.0	10.92

TABLE VI

QUANTITIES OF PRINCIPAL CONSTITUENTS IN GRAMS PER CAPITA DAILY. SEWAGE OF SMALL CITIES IN MASSACHUSETTS, CONTAINING SOME TRADE WASTE.

(Calculated from table on page 6.)

		Grams per Capita.						
		Brockton.	Framingham.	Gardner.		Marlboro.	Westboro.	Average.
				Gardner.	Templeton.			
Flow per person connected with sewer, U. S. G.		35	87	86	55	110	94	77.8
Total solids	Total	115	194	125	90	187	132	149
	Fixed	43	95	50	34	96	56	68
	Volatile	72	99	75	56	91	76	81
Suspended solids	Total	55	70	50	42	57	66	53
	Fixed	7	16	8	5	9	17	9
	Volatile	48	54	42	37	48	49	44
Dissolved solids.	Total	60	123	75	48	130	66	96
	Fixed	36	78	42	29	87	39	59
	Volatile	24	45	33	19	43	27	37
Nitrogen as	Free NH_3	4.2	8.6	6.6	5.7	10.8	4.9	7.7
	Alb. NH_3	1.2	2.1	1.6	1.4	1.8	1.6	1.6
Oxygen consumed in 5 minutes		21	16	16	13	19	13	19.6
Chlorine		13	23	11	9	25	8	17.7

TABLE VII

QUANTITIES OF PRINCIPAL CONSTITUENTS IN GRAMS PER CAPITA DAILY. SEWAGE OF ENGLISH CITIES, CONTAINING TRADE WASTE

(Calculated from table on page 8.)

		Grams per Capita.					
		Birmingham.	Bradford.	Leeds.	Leicester.	Manchester.	Sheffield.
Flow per person connected with sewer, U. S. G		36	65	45.4	47.4	52.8	48
Total solids	Total	266	652	317	326	247	.....
	Fixed	.....	437	.....	.....	118	.....
	Volatile	.....	215	.....	.....	129	.....
Suspended solids	Total	98	207	133	123	65	76
	Fixed	.....	65	.....	.....	32	31
	Volatile	.....	142	.....	.....	33	45
Dissolved solids	Total	168	445	184	203	182	.....
	Fixed	.....	372	.....	157	86	.....
	Volatile	.....	73	.....	46	96	.....
Nitrogen as	Free NH_3	5.68	11.7	3.9	8.3	5.8	5.8
	Total alb. NH_3	2.21	8.2	1.4	2.8	1.4	1.64
Oxygen consumed in 4 hrs.	Unfiltered	35.3	49.8	17.0	22.1	22.7	14.5
	Filtered	18.9	.....	.....	.....	.....	.....
Chlorine		28	36.7	28.4	23.0	40	22.6

Chemical Composition and Properties of Organic Compounds Occurring in Sewage

As has been previously stated, the organic matter in sewage is made up chiefly of urea, proteids, carbohydrates (including cellulose and woody fiber), fats and soap. These substances, chiefly through the action of bacteria, undergo greater or less decomposition, and to follow the changes which take place it is necessary to have a knowledge of the composition and properties of these substances.

Urea, the chief constituent of urine, is a compound of carbon, hydrogen, oxygen and nitrogen, having the formula $CO(NH_2)_2$. It is a white crystalline substance, soluble in water, and is readily converted into ammonium carbonate; so quickly does this change take place that undecomposed urea is rarely found in sewage at an outfall sewer. The odor of ammonia that so often pervades the air in the neighborhood of urinals and barnyards is chiefly due to the decomposition of urea.

The Proteids. The proteids, or albuminoid substances, form the principal constituents of the animal organism. They are also found in the living parts of plants, particularly in the seeds. The number of known proteids is large, and they are sometimes divided into groups according to their animal or vegetable origin; but between the members of the different groups there are no very striking chemical differences. They all contain carbon, oxygen, hydrogen and nitrogen; many contain sulphur, some iron and phosphorus.

In their coagulated state proteids are white amorphous substances, some of which, like egg albumen, are soluble in water, while others, like globulin, are insoluble. Most of them are soluble in dilute mineral acids, all are soluble in concentrated acetic and phosphoric acids. In chemical composition they differ very slightly, as is shown by the percentage composition of three of the chief proteids:

	Albumin.	Fibrin.	Casein.
C..........	53.5	52.7	53.8
H..........	7.0	6.9	7.2
N..........	15.5	15.4	15.6
O..........	22.4	23.8	22.5
S..........	1.9	1.2	0.9

The molecular structure is highly complex, the number of atoms in a single molecule being generally considered to reach into the thousands.

The Carbohydrates. The term "carbohydrate" is applied to a large class of compounds widely distributed in nature. The molecule of the common carbohydrates contains six, or a multiple of six, carbon atoms; and the ratio of their hydrogen to their oxygen atoms is the same as that of these elements in water, two of hydrogen to one of oxygen. The carbohydrates include starches, sugars, cellulose and wood fiber. The general composition of these compounds is shown by the formulæ:

Starches, $(C_6H_{10}O_5)_x$.

Sugars, $\left\{\begin{array}{l}\text{Glucose, } C_6H_{12}O_6. \\ \text{Saccharose, } C_{12}H_{22}O_{11}.\end{array}\right.$

Cellulose, $(C_6H_{10}O_5)_y$.

Some, like sugars, are soluble in water; others, like the starches, are insoluble. The starches and the cellulose can be converted into sugars by dilute acids and by certain organic ferments. The sugars can be changed into alcohol by yeast and certain enzymes, carbon dioxide being evolved.

The Fats. The fats are compounds of the triatomic alcohol, glycerol, combined with fatty acids, such as stearic, palmitic and oleic acids. These compounds are known as glycerides. The special names of the most important fats are stearine, palmatine, oleine and butyrine, and their chemical composition is as follows:

Stearine, $C_3H_5(C_{18}H_{35}O_2)_3$.
Oleine, $C_3H_5(C_{18}H_{33}O_2)_3$.
Palmatine, $C_3H_5(C_{16}H_{31}O_2)_3$.
Butyrine, $C_3H_5(C_4H_9O_2)_3$.

Most animal fats are a mixture of two or more of these substances in different proportions; and some, as, for example, butter, contain glycerides of other fatty acids.

Of the most important glycerides, stearine and palmatine are solids, oleine and butyrine liquids. They are insoluble in water, but soluble in alcohol. Chemically they consist of carbon, hydrogen and oxygen, the percentage composition depending upon the particular kind of fat. Their molecular structure is not complicated, but they are among the more stable of organic compounds, and, outside of the human body, are not easily broken down by bacteria. They are, however, acted upon by mineral acids, giving glycerol and a fatty organic acid; and they are decomposed by the alkalies into glycerol and the alkali salts of the fatty acids, compounds known under the name of soaps.

Soap. Soap is the generic name given to the mineral salts of the fatty acids; and soaps are formed when fatty acids or fats are treated with basic hydroxides. The most common soaps, whether hard or soft, are to-day made from sodium. Such soaps, being universally used, occur in solution in considerable quantities in alkaline sewage, and, being only slightly acted upon by bacteria, undergo very slight change as long as the sewage remains alkaline. When the sewage becomes acid, they are decomposed into insoluble fatty acids and soluble salts of sodium — or potassium in the case of old-fashioned soft soap.

The Biolysis of Sewage. The term "biolysis of sewage" is used to express the breaking down, or the decomposition, of animal and vegetable substances, and is usually divided into two stages, the first called putrefaction, and the second or final stage, oxidation or nitrification. This decomposition of organic matter is chiefly brought about by bacteria, or by substances formed by bacteria, the so-called enzymes. In the first or putrefactive stage, the active agents are mainly bacteria which live and multiply in the absence of air. In the second stage the active agents are bacteria which require oxygen.

Rideal, who has made a very careful study of the changes that

take place during biolysis, divides the process into four stages and summarizes the order of the changes as follows (Rideal, 1906):

	Substances dealt with.	Characteristic Products.
Initial. Transient aerobic changes by the oxygen of the water supply rapidly passing to —	Urea, ammonia, and easily decomposable matters.	
First Stage. Anaerobic liquefaction and preparation by hydrolysis.	Albuminous matters. Cellulose and fiber. Fats.	Soluble nitrogenous compounds. Phenol derivatives. Gases. Ammonia.
Second Stage. Semi-anaerobic breaking down of the intermediate dissolved bodies.	Amido-compounds. Fatty acids. Dissolved residues. Phenolic bodies.	Ammonia. Nitrites. Gases.
Third Stage. Complete aeration: oxidation and nitrification.	Ammonia and carbonaceous residues.	CO_2, H_2O, and nitrates.

Taking Rideal's summary as an outline of what takes place, the changes that are thus brought about by the action of micro-organisms on proteids, carbohydrates and fats, may be considered a little more in detail.

1. Proteids. In the first of Rideal's stages, the proteids are broken down into the albumoses and peptones, with the separation of the sulphur as hydrogen sulphide, or as mercaptans. The albumoses and peptones have a less complex molecular structure than the proteids; they are soluble in water and are not coagulated by heat. These compounds, also during the first stage, break down into the so-called amido acids, chiefly acids of the fatty hydrocarbon series containing carbon, hydrogen, oxygen and nitrogen in the form of the amido group, (NH_2).

The amido acids thus formed are decomposed during the first and second of Rideal's stages, giving ammonia, phenols, fatty and aromatic acids. All of the nitrogen, however, is not converted into ammonia, for part remains united to hydrogen and carbon, forming amines like trimethyl amine, $(CH_3)_3N$, part is liberated as nitrogen, while a certain portion is undoubtedly converted directly into nitrous acid.

The last change in the process of decomposition is a partial or complete oxidation of the organic substances formed by the decomposition of the amido acids, resulting in the production of water, carbon dioxide, nitrous and nitric acids.

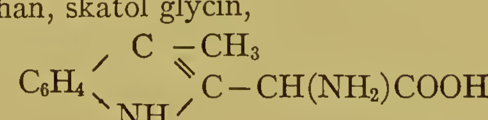
The principal products of the bacterial action on the proteids are amido acids of the fatty hydrocarbon series, relatively small amounts of the aromatic compounds, such as phenyl alanin, tyrosin, tryptophan, phenol, skatol and indol, being formed.

Taking the three principal proteids, albumin, fibrin and casein, the following table represents some of the principal changes that take place when proteids are acted upon by the bacteria contained in sewage:

Albumin, Fibrin, Casein } Yield

Glycin, amido-acetic acid,	$CH_2(NH_2)COOH$
Leucin, amido-isocaproic acid,	$C_5H_{10}(NH_2)COOH$
Tyrosin, β-oxyphenyl-amido-propionic acid	$CH_2C_6H_4OH$ $CH(NH_2)COOH$
Aspartic, amido-succin-amic acid	CH_2COOH | $CH(NH_2)COOH$
Asparagin, amido-succin-amic acid	$CH_2CO(NH_2)$ | $CH(NH_2)COOH$
Glutaminic, α-amido-glutaric acid	$CH_2 \begin{cases} CH(NH_2)COOH \\ CH_2COOH \end{cases}$
Cystin, α-diamido β-dithio dilactic acid	$\begin{array}{lcl} CH_2-S-S-CH_2 \\ \vert & & \vert \\ CH(NH_2) & & CH(NH_2) \\ \vert & & \vert \\ COOH & & COOH \end{array}$

Tryptophan, skatol glycin,



Further changes take place as follows:

Glycin into ammonia and acetic acid.

Leucin into ammonia and isocaproic acid.

Tyrosin into phenol and aromatic acids.

Aspartic into ammonia and malic acid, then succinic.

Asparagin into ammonia and succinic acid.

Glutaminic into ammonia and probably succinic acid.

Cystin into hydrogen sulphide, ammonia, and propionic acid.

Tryptophan into skatol and indol.

2. Carbohydrates. Sugar and starches are very easily hydrolized and broken down by the action of bacteria, and though alcohol may be one of the products of the decomposition, the principal substances formed seem to be butyric acid, lactic acid, water, carbon dioxide and hydrogen. Cellulose and woody fiber are also broken down and liquefied, but comparatively slowly, and the action is often so sluggish that they are but little changed in the process of sewage treatment. Hydrolysis plays an important part, at least during the first stages of the decomposition of cellulose, and the products formed are undoubtedly similar to those produced from the sugars and starches.

3. Fats. The decomposition of fats, brought about by the action of bacteria and molds, is again largely a process of hydrolysis, the fatty acids, stearic, palmitic, oleic, butyric and glycerol, being the chief products. The fatty acids are then further resolved into carbon dioxide, hydrogen and marsh gas. The breaking down of the fats takes place very much more slowly than that of the proteids, and it appears probable that they must first emulsify before being acted upon by bacteria. This emulsification is at least partly brought about by the ammonia set free in the decomposition of the amido acids. The slow decomposition of fats and greases makes a sewage containing large amounts of these substances very much more difficult to purify than a sewage where the organic matter is mainly in the form of proteids and carbohydrates.

In the decomposition of proteids, carbohydrates and fats, fatty organic acids, as has been stated, are always formed, and these acids are further broken down, giving as a final result water, carbon dioxide, hydrogen and methane. The following table, taken from Rideal (1906), gives in a concise form the products formed by the decomposition of the principal fatty organic acids:

TABLE OF FERMENTATION OF ORGANIC SALTS

(For simplicity, the sodium salts are taken, though the lime salts are rather more fermentable.)

Salt Fermented.	Products.
Formate	Acid sodium carbonate, $NaHCO_3$, carbonic acid and hydrogen.
Acetate	Acid sodium carbonate, $NaHCO_3$, carbonic acid, and methane, CH_4.
Lactate. Undergoes four different fermentations.	1. Propionic acid, and as by-products, acetic and succinic acids, and alcohol. 2. Propionic and valeric acid. 3. Butyric and propionic acid. 4. Butyric acid and hydrogen.
Malate. Different fermentations.	1. Chief product, propionic acid; by-product, acetic acid. 2. Chief product, succinic acid; by-product, acetic acid. 3. Butyric acid and hydrogen. 4. Lactic acid and CO_2.
Tartrate	1. Chief product, propionic acid; by-product, acetic acid. 2. Butyric acid. 3. Chief product, an acetate; by-products, alcohol, butyric and succinic acids.
Citrate	Acetic acid in large quantities, with small quantities of alcohol and succinic acid.
Glycerate	1. An acetate, with small quantities of succinic acid and alcohol. 2. Formic acid, with some methyl alcohol and acetic acid.

The propionic, butyric, succinic and valeric acids under active microbic action, as well as any amines of these acids

which are formed as products of the decomposition of the salts of the more complex acids, are also eventually broken down, giving, as the final result, marsh gas, hydrogen, carbon dioxide, ammonia and water.

These statements regarding the biolysis of sewage can only be regarded as general, since many other substances besides those noted are undoubtedly formed. Thus in the decomposition of cellulose we know that it slowly undergoes liquefaction, but we know little, if anything, regarding the process or the substances formed. During the decomposition of proteids the odors which are given off are not due alone to hydrogen sulphide, the mercaptans and the amines, but to other substances regarding which we have practically no knowledge. Further, in the changes that have been noted, there is always formed a comparatively large amount of organic matter which most effectively resists further change, resembling in its properties the humus of the soil. What this substance is, or how it is produced, we have little, if any, idea.

Though much remains to be worked out before we can state exactly what takes place during the biolysis of sewage, the essential facts are as follows: urea is decomposed, ammonium carbonate being formed; the proteids are first broken up into peptones, then into amido acids, these amido acids being resolved into nitrogen, amines, ammonia and the fatty acids; the fats yield glycerol and fatty acids; the carbohydrates yield chiefly acids like lactic and butyric, though some alcohol may be formed; and the fatty acids are to a greater or less extent decomposed into hydrogen, marsh gas, carbon dioxide and water. The sulphur of the proteids is changed into hydrogen sulphide or into mercaptans, and the amines and ammonia set free from the amido acids are oxidized to nitrous and nitric acid. During these changes a certain amount of resultant organic matter is formed, similar to the humus of the soil.

The important question in sewage treatment is, — How can these changes be brought about without causing offense, and at the least cost?

CHAPTER II

DISPOSAL OF SEWAGE BY DILUTION

The Disposal of Sewage by Discharge into Water. Direct discharge into the nearest body of water is still the commonest method of sewage disposal in the United States; and it proves in many cases a fairly satisfactory one. Where the body of water is of sufficient size, the sewage quietly disappears without nuisance and without permanent injury to the lake or stream. Thus Weston estimates that the Mississippi River above New Orleans receives a billion and a half gallons of sewage per day, amounting to a flow of 2310 cubic feet per second. Yet at New Orleans the river is no more polluted than any surface stream.

This method of dealing with sewage is commonly called "Disposal by Dilution," and the term is a fairly descriptive one. Impurities are of course just as truly present whether they are distributed in a large body of water or a small one. From a practical standpoint, however, dilution amounts to the same thing as removal, and may reasonably be considered to constitute real purification. If a small stream which contains one typhoid germ in every tumblerful of water mixes with an unpolluted stream of a hundred times its volume, the danger of contracting typhoid fever by drinking a glass of water is diminished to one per cent of what it was before.

Beyond the effect of dilution, pure and simple, there are other processes of purification at work in a stream, which ultimately lead to the oxidation of unstable organic compounds and the destruction of pathogenic bacteria. These processes of chemical and bacterial self-purification are quite distinct from each other and must be considered separately. Both, however, are indirectly favored by dilution; and dilution itself, aside from any chemical or biological agencies, often plays a predominant

part. In order to be convinced of this it is only necessary in studying the analyses of a purifying stream to compare the reduction of chlorine with the diminution of bacteria and organic matter. It will generally appear that nine-tenths of the improvement manifest is due to dilution alone.

Self-Purification of Streams. There are two principal factors which mainly control the removal of organic polluting material from water, — the direct physical effect of sedimentation, and the chemical changes in the organic matter itself, which are set up mainly under the action of aerobic bacteria. The primary force at work in freeing the water from suspended impurities is no doubt the force of gravity, and this alone accounts for a large part of the purification which ensues in streams or ponds. Gravity alone, however, if unaccompanied by any other action, would only localize the trouble on the bottom, and might even accentuate it by the very fact of concentration. Where suspended impurities gather too rapidly, sludge banks often accumulate to considerable dimensions, and instead of self-purification a gross nuisance may result.

True organic self-purification implies chemical changes, of such a nature as to alter the decomposable organic matter to a stable form. This is brought about by a number of agencies. Direct chemical changes play a certain part, as, for example, in the simplification and oxidation of sulphur and iron compounds. According to Adeney (Letts and Adeney, 1908), such direct chemical oxidations are of very considerable significance. Algæ also play a part. Bokorny (1897) and various observers attribute much importance to the activity of the Green Algæ; but Chick and others (Brezina, 1906) have shown that while these organisms do consume free ammonia, they form neither nitrites nor nitrates and ultimately yield up the nitrogen which they have absorbed in the form of albuminoid ammonia. Some investigators have laid stress upon the consumption of organic matter by mold-fungi (like *Leptomitus* and *Beggiatoa*), by Protozoa, worms, insects, Crustacea, mollusks and fishes. These organisms, too, while they temporarily alter the condition of their food

material, do not render it on the whole any more stable or less putrescible. The only forms which we know to be able to effect extensive simplifications and oxidations of organic matter are certain bacteria. It is most probable that the most active agents in the chemical self-purification of streams are microbes of the same types as those which in sewage filters change relatively large amounts of organic matter, more or less quantitatively, into the mineral form.

There is at any rate no doubt that the process of self-purification, as far as the organic matter is concerned, — by whatever agents it may be carried out, — is essentially a process of oxidation, in the course of which substances of the free ammonia and albuminoid ammonia classes are changed to nitrites and eventually to nitrates.

Dr. Adeney (Letts and Adeney, 1908) in a series of elaborate and beautiful experiments has shown that the process of self-purification may be divided into two phases. In the first stage carbonic acid, water, ammonia and humus-like organic compounds are formed. The oxidation of the carbon atoms in the molecule is the particular characteristic of this phase. In the second stage the humus matters and the ammonium compounds are further changed with the final production of nitric acid, as well as carbonic acid and water. This is pre-eminently the phase of nitrogen oxidation. The final change may be very slow, and the relations between the substances involved indicate that it is highly complex, the ammonium compounds not being readily oxidized in the absence of the humus materials.

The most delicate measure of the process of chemical self-purification is the change in oxygen content. As self-purification actively proceeds oxygen is rapidly used up, and as the process slackens it is reabsorbed and gradually returns to a normal value. The whole history of the pollution and self-purification of streams may be traced by the diminution and gradual restoration of this constituent. Dibdin's studies of the Thames below London are most significant in this respect and illustrate on a practical scale the enormous volumes of the oxidizing

agent needed. He estimates (Dibdin, 1904) that 2000 tons of oxygen are absorbed by the river between Teddington and Southend in this process. The proportion of dissolved oxygen, expressed as "per cent of saturation," at various points along the river on the high tide, is plotted in the diagram below by

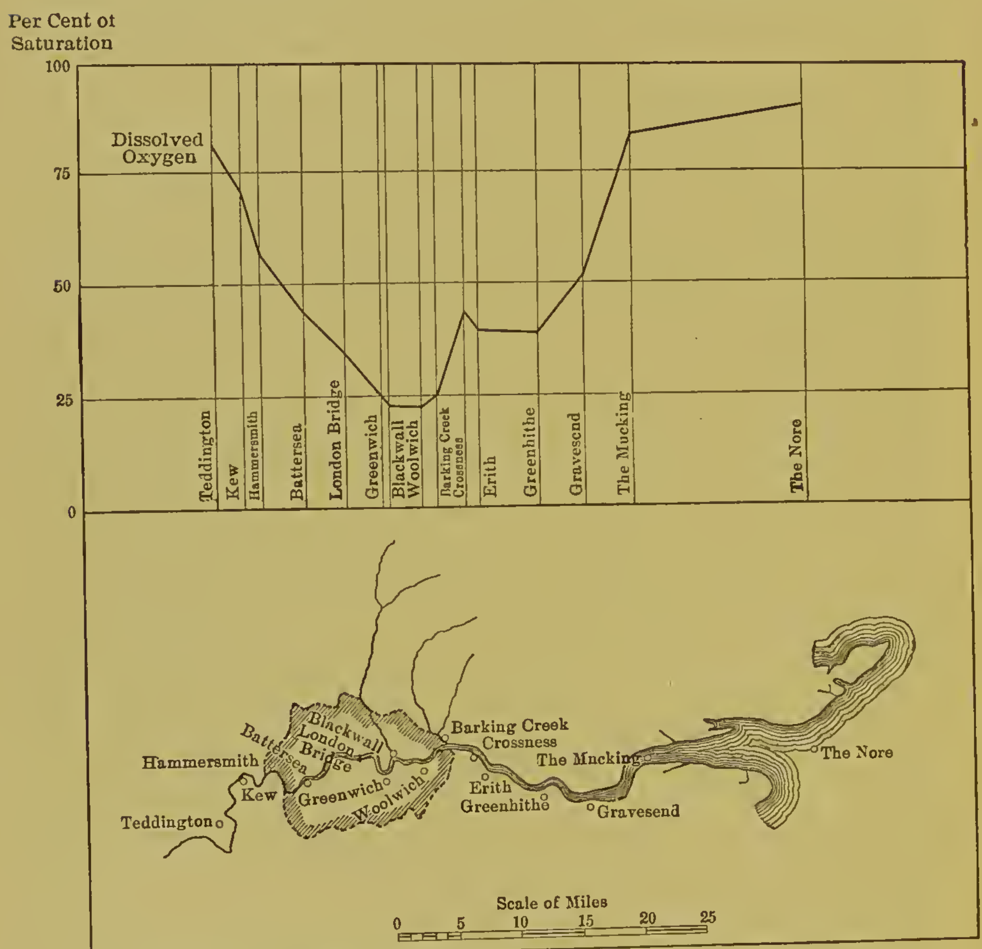


FIG. 2. Dissolved Oxygen Changes in the Thames at London.

Winslow and Phelps (1906) from figures given by Dibdin (1904) for 1893–94. As the river enters the city between Kew and Battersea its oxygen content falls from 70 per cent to 43 per cent, and the progressive pollution continues until at Woolwich the oxygen value is only one-fifth that of saturation. Below Barking Creek the heavy pollution ceases, absorption

of oxygen overbalances its consumption, and the normal conditions are gradually restored. The same general relations are shown in Table VIII, quoted by the Connecticut State Sewage Commission (1899). The ratio of oxygen to nitrogen, which changes from 1 : 2 at Kingston, above London, to 1 : 62 at Greenwich, is most significant.

TABLE VIII

DISSOLVED GASES IN THE THAMES ABOVE AND BELOW LONDON, ENGLAND (CONNECTICUT, 1899)

Analyses by Roscoe and Schorlemmer. (Cubic centimeters per liter.)

	Kingston	Hammersmith.	Somerset House.	Greenwich.	Woolwich.	Erith.
Total volume of gas	52.7	....	62.9	71.25	63.05	74.3
Carbon dioxide	30.3	....	45.2	55.6	48.30	57.0
Oxygen	7.4	4.1	1.5	.25	.25	1.8
Nitrogen	15	15.1	16.2	15.4	14.5	15.5
Ratio of oxygen to nitrogen	1 : 2	1 : 3.7	1 : 10.8	1 : 62	1 : 5.8	1 : 8.6

Sometimes the progressive change in oxygen on the one hand and in oxidizable organic matter on the other hand may accurately record even very slight changes in the physical conditions of the process. A good example of this is the diagram reproduced below from data collected by Woodman, Winslow and

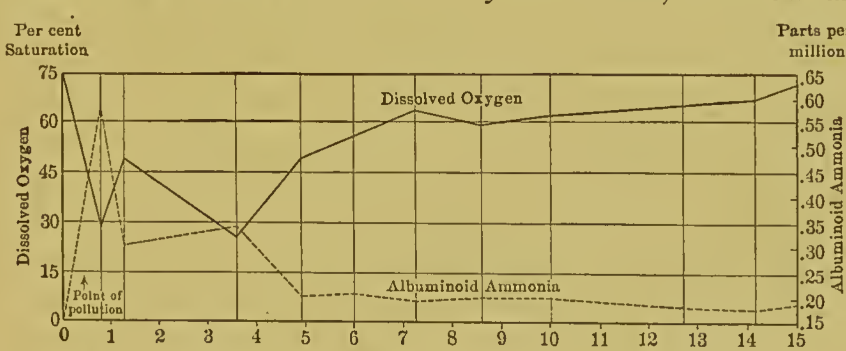


FIG. 3. Pollution and Self-Purification in the Sudbury River.

Hansen (1902). The Sudbury River is heavily polluted at Saxonville (marked Point of pollution on the diagram) by the wastes from a woolen mill. Between the one- and two-mile sampling points a large spring of pure water enters the river

bed and causes the dissolved oxygen to rise. From this point for three miles the flow of the stream is rapid and self-purification slow. The river then enters an area of meadows, where it winds along through a weedy channel at a rate of not more than one-fourth mile an hour. Self-purification here goes on actively, so that three miles below the entrance to the meadows and six miles below the mill the chemical constituents of the stream fall to their normal. The striking thing about the diagram is the close inverse relation of the curves for oxygen and albuminoid ammonia, which illustrates the essential nature of self-purification as a chemical oxidation of putrescible organic matter.

Adeney (Letts and Adeney, 1908) has shown that the great bar to oxygenation lies not in slow diffusion but in the fact that absorption of oxygen from the air by a quiet surface is very gradual. Columns of water six feet long are quickly aerated when the surfaces of the columns are broken by mechanical means. In this respect, therefore, rapidly running water has a distinct advantage over that which is stagnant; and wind action may have an important effect in promoting purification.

The bacterial self-purification of streams is an entirely separate and independent process from that which has just been considered. While the organic matter of the sewage is being digested and mineralized by the water bacteria, the original sewage bacteria are dying out and disappearing. Sedimentation removes them from the flowing liquid, with the solid particles to which so many of them are attached. Light, low temperature, predatory micro-organisms of the stream, excess or deficiency of oxygen, osmotic conditions to which they are unadapted, and above all the lack of the rich food supply to which they are accustomed, — all these conditions are inimical to the sewage forms.

It is difficult to estimate the exact relative importance of these various factors in self-purification. Taken together they may be grouped under the general term, the environment. River water is an unfavorable environment for many bacteria; and

foreign bacteria introduced into sewage, microbes whose home is the alimentary canal, are unable to adapt themselves to it, and die. The process of elimination is gradual, since different species of bacteria and even different individuals of the same species are differently affected. The reduction is greater, however, the longer the process goes on, and for this reason time is the most important of all conditions for bacterial purification. It used to be said that "running water purifies itself;" but this is just the reverse of the truth. It is stagnant water which purifies itself, for storage is the controlling factor in self-purification.

Self-Purification in the Des Plaines and Illinois Rivers. The most careful study of self-purification which has ever been conducted on a large scale was carried out in connection with the discharge of the sewage of Chicago into the Illinois River; and the results obtained in this investigation serve well to illustrate the general principles involved. The facts of the case in outline were as follows. The sewage of the city of Chicago prior to 1900 was mainly discharged into the south branch of the Chicago River, a sluggish stream flowing east into Lake Michigan. In order to prevent gross pollution of the lake which supplies water to the city, pumps were installed thirty years ago to pump the water of the south branch across a narrow divide through the Illinois and Michigan canal into the Des Plaines River, flowing to the southwest. At times of heavy rain the pumps failed to cope with the natural current of the stream. A new canal was therefore constructed at a cost of nearly forty million dollars to effect a permanent connection between Lake Michigan and the Chicago River on the one hand and the Des Plaines River on the other. This drainage canal is designed to carry a flow of 600,000 cubic feet per minute from Lake Michigan plus the sewage in the Chicago River and the Illinois and Michigan canal. The general relations of the drainage area are indicated in Fig. 4. The Des Plaines enters the Illinois River below Joliet and the Illinois is later further diluted by the Kankakee, the Fox, the Big Vermilion and the

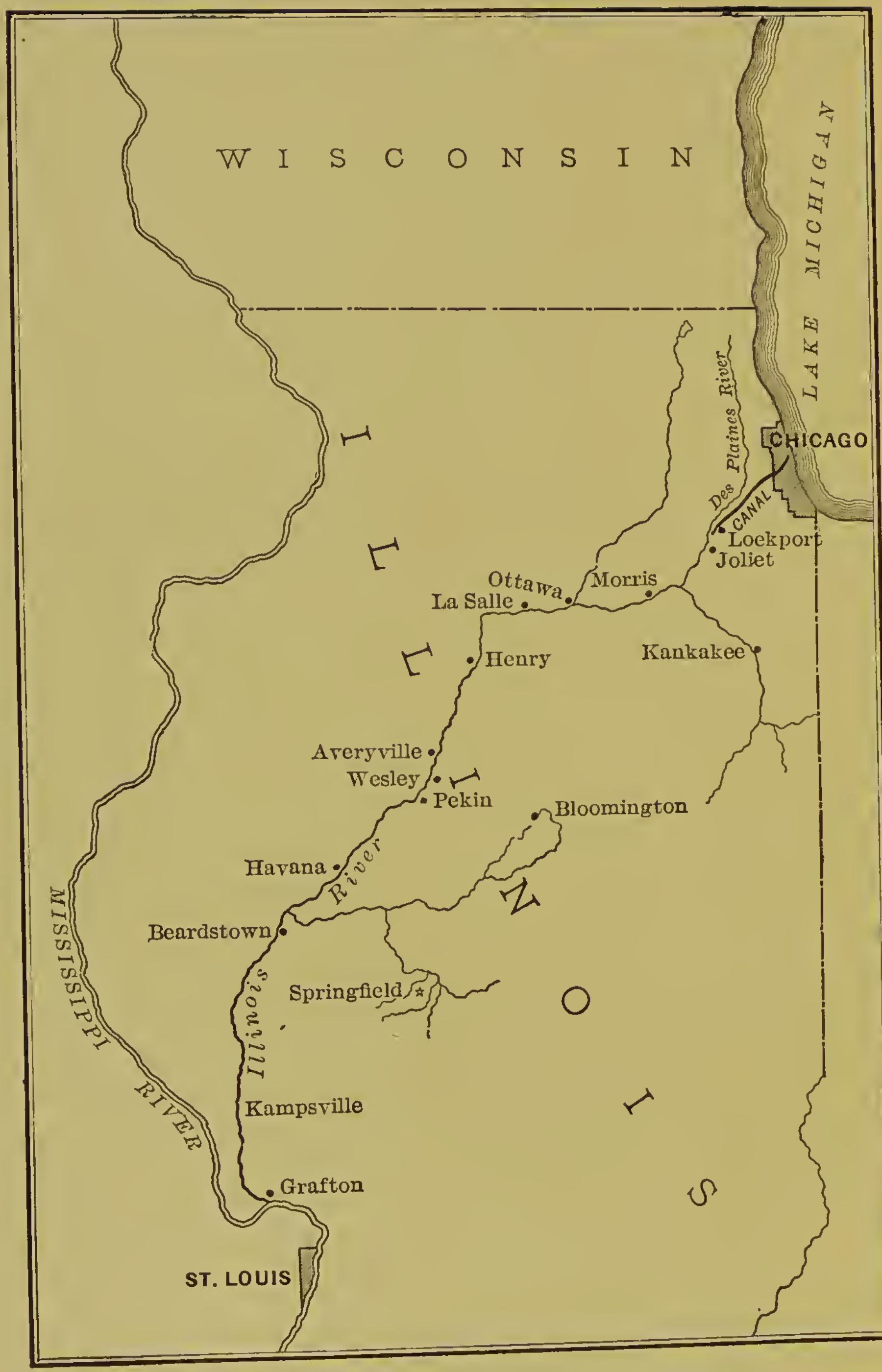


FIG. 4. Chicago Drainage Canal and Illinois River.

Sangamon rivers. At Grafton the Illinois enters the Mississippi. A short distance below, the water of the Mississippi is used as a source of water supply by the city of St. Louis; and on the day the canal was opened, Jan. 17, 1900, the State of Missouri instituted proceedings before the Supreme Court of the United States, praying for an injunction against the State of Illinois and the sanitary district of Chicago. The leading sanitary experts of the country testified in the case, and the complete records, which occupy 8000 printed pages, have been digested and published in brief form by the U. S. Geological Survey (Leighton, 1907).

The case was an ideal one for the study of self-purification. The sewage of a large city, carrying four million pounds of urine and fecal matter per day, was discharged into a stream which gave at first a dilution of about one part sewage in ten parts of water. This polluted stream with successive dilutions from purer rivers flowed for a distance of 357 miles in an average time, variously estimated at eight to eighteen days.

TABLE IX

SELF-PURIFICATION IN THE DES PLAINES AND ILLINOIS RIVERS

January–June, 1900.

Station.	Period of Flow, Days.	Parts per Million. Chlorine.	Free Ammonia.	Albuminoid Ammonia.	Nitrites.	Nitrates.	Bacteria per c.c.
Illinois and Michigan Canal, Bridgeport	......	96.6	8.05	2.05	.021	.074	631,000
Illinois and Michigan Canal, Lockport	1.6	124.5	10.90	2.07	.013	.066	1,755,000
Des Plaines R., Joliet	1.7	41.5	4.22	.83	.021	.086	744,286
Illinois R., Morris	2.5	24.5	2.46	.60	.075	.424	445,000
Illinois R., Ottawa	3.4	15.3	1.55	.41	.197	.966	116,000
Illinois R., La Salle	......	17.5	1.05	.43	.109	.979	94,000
Illinois R., Henry	5.3	13.3	.92	.38	.102	.800	64,200
Illinois R., Averyville	......	13.5	.81	.37	.004	1.150	51,800
Illinois R., Wesley	......	12.0	.56	.41	.083	1.030	36,800
Illinois R., Pekin	9.9	12.3	.70	.43	.060	.990	68,400
Illinois R., Havana	11.4	11.2	.60	.36	.065	.570	23,100
Illinois R., Beardstown	12.8	10.7	.69	.44	.106	.685	28,200
Illinois R., Kampsville	15.5	11.3	.66	.44	.044	.870	33,700
Illinois R., Grafton	17.7	9.8	.46	.42	.031	1.060	21,000

The most important features of the resulting self-purification are indicated in Table IX. The estimates for the period of flow are taken from the testimony of Isham Randolph, C.E. According to the experts on the St. Louis side these figures should be cut in half. The analytical data are from the testimony of Prof. E. O. Jordan.

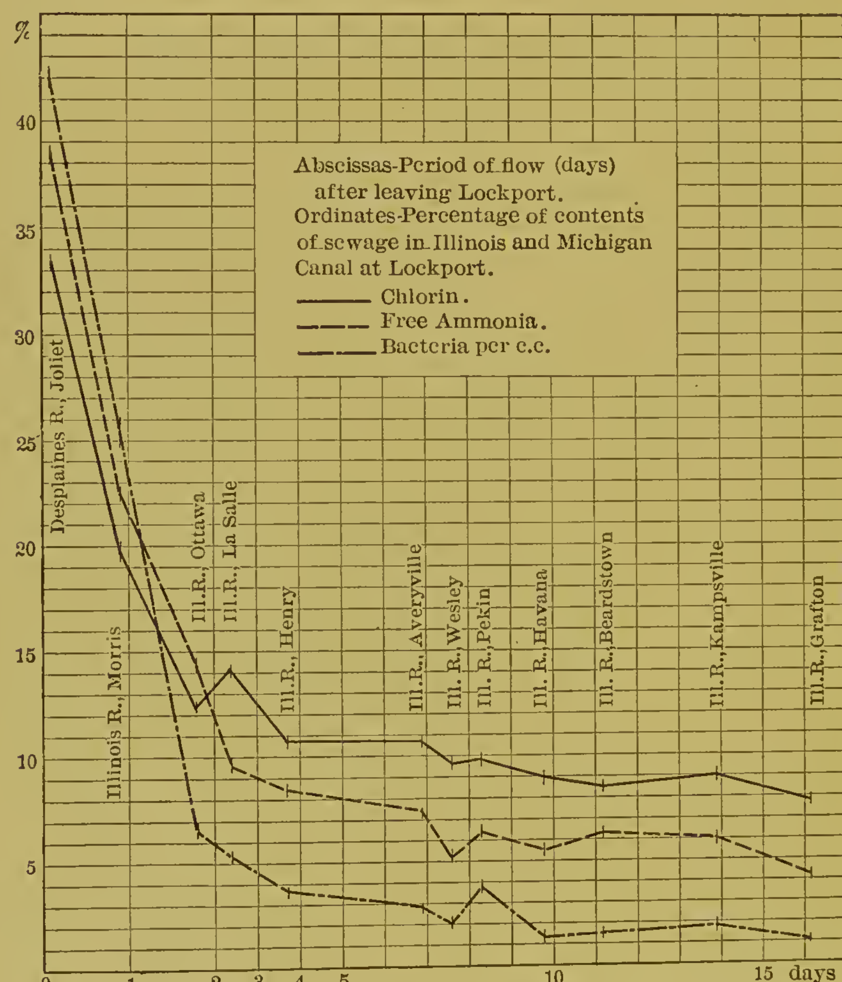


FIG. 5. Self-Purification in the Des Plaines and Illinois Rivers.

The reduction of chlorine, plotted in the diagram above, measures pretty closely the amount of self-purification by dilution alone. From the Illinois and Michigan canal at Lockport

to the Des Plaines River at Joliet it falls to one-third its former value, as a result of the dilution furnished by the drainage canal. The further progress of the curve shows a fairly steady decrease, the final value reached being about eight per cent of the value at Lockport.

The changes in organic matter during the same period of flow are indicated by the curve for free ammonia, in Fig. 5. The decrease of free ammonia parallels pretty closely the decrease in chlorine; but it is more marked. The final free ammonia value at Grafton is only four per cent of the value at Lockport. Here, as in the case of chlorine, it is probable that dilution is responsible for most of the improvement; but some of the free ammonia not affected by dilution has been removed by oxidative processes. The nitrite and nitrate figures show the progress of the changes which are at work. Where the process is most active, from Ottawa to Henry, nitrites rise to a maximum. From Ottawa to Wesley nitrates are increasing. At Pekin, the discharge of sewage from Peoria causes some temporary reducing changes, followed again by an increase of nitrates at Kampsville and Grafton.

The bacterial self-purification which accompanies these chemical changes is also indicated in Fig. 5. A steady decrease is apparent from Lockport to Wesley. At Pekin there is a notable increase, due to the sewers of Peoria, followed by a second self-purification. Comparison of the total improvement effected shows a bacterial reduction of nearly ninety-nine per cent from Lockport to Grafton. The reduction here is considerably greater than in the case of the organic constituents. Aside from dilution, true self-purification has played a part.

It is interesting to notice, as epidemiological evidence of the value of self-purification, that no striking excess of typhoid fever at St. Louis followed the opening of the Chicago drainage canal. The experts for the plaintiff were able to show a slight increase in typhoid fever; but this might well have been due to the pollution of the Mississippi River from sources other than Chicago, or to causes other than water. The Supreme Court

finally dismissed the case without prejudice, on the ground that damage to St. Louis was not proved.

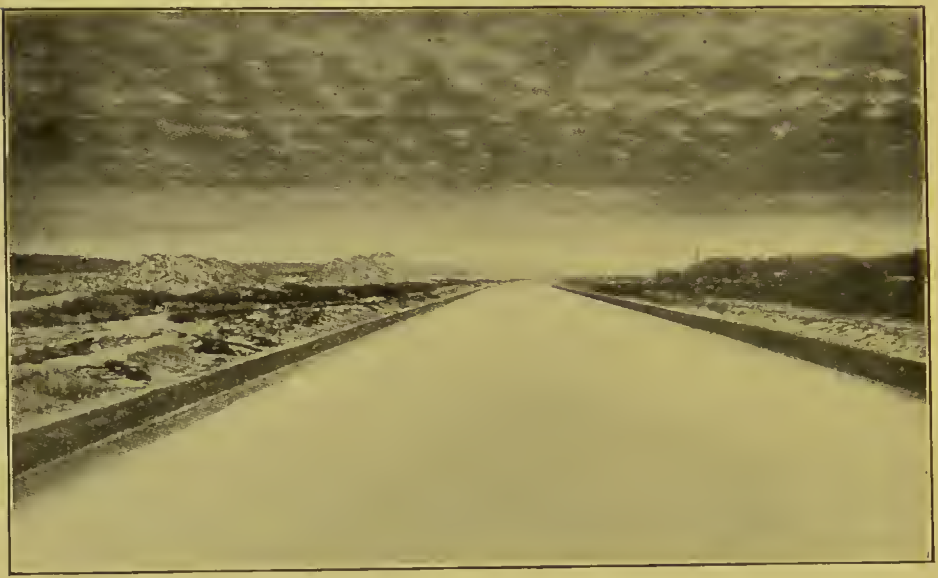


Fig. 6. View of the Chicago Drainage Canal at Willow Springs (courtesy of Robert R. McCormick).

The Critical Point in Stream Purification. In order that the purifying process may proceed successfully it is obviously essential that sufficient oxygen should be present for the oxidation of the organic matter. The amount of this oxygen is by no means inconsiderable. Dibdin (1903), as shown in the table below, estimates it at from one to three times the weight of the organic substance to be acted upon for complete oxidation to the mineral form.

TABLE X

PARTS OF OXYGEN REQUIRED TO OXIDIZE ONE PART OF VARIOUS ORGANIC SUBSTANCES (DIBDIN, 1903)

Substance.	Oxygen required.				Oxygen already present.	Difference, or additional oxygen required for complete oxidation.
	By the nitrogen.	By the hydrogen.	By the carbon	Total.		
Gelatin	0.523	0.528	1.333	2.384	0.251	2.133
Chondrin	.411	.568	1.310	2.289	.294	1.995
Albumen	.457	.568	1.414	2.439	.220	2.219
Cellulose, woody fiber	......	.496	1.184	1.680	.494	1.186
Starch	......	.496	1.184	1.680	.494	1.186
Fat, stearic acid	......	1.016	2.025	3.041	.113	2.928

When a gradually increasing amount of sewage is added to a stream the organic matter will be cared for as long as oxygen is present in the water, either free or in easily reducible forms. Additional pollution only makes the process somewhat slower. At a certain point, however, if the increase of pollution continues, the oxidations which are set up will consume all the available oxygen more rapidly than it can be renewed by diffusion from the surface. This is a critical point. Once passed, the whole process changes. Instead of oxidations, a new series of bacterial changes are set up, — reducing actions which partially break down the organic compounds into less complex but still unoxidized bodies. This process leads to no final condition of stability, but to a progressive accumulation of half-decomposed organic matter; and it is accompanied by the production of foul-smelling gases, hydrogen sulphide, amines, mercaptans, etc. Putrefaction has taken the place of purification, and a stream has been converted into a septic tank or open cesspool.

The Thames below London is a classic example of a stream polluted beyond its Critical Point. Budd in his monograph on typhoid fever (Budd, 1873) thus describes the conditions during the hot months of 1858 and 1859:

" Stench so foul, we may well believe, had never before ascended to pollute this lower air. Never before, at least, had a stink risen to the height of an historic event. Even ancient fable failed to furnish figures adequate to convey a conception of its thrice Augean foulness. For many weeks, the atmosphere of Parliamentary Committee-rooms was only rendered barely tolerable by the suspension before every window of blinds saturated with chloride of lime, and by the lavish use of this and other disinfectants. More than once, in spite of similar precautions, the law courts were suddenly broken up by an insupportable invasion of the noxious vapor. The river steamers lost their accustomed traffic, and travelers, pressed for time, often made a circuit of many miles rather than cross one of the city bridges."

Another famous example is that of the Seine below Paris, before the purification of the sewage of the city. The river after the entrance of the main sewers is described as black and stinking. For a long distance it was in active fermentation, and in some places gas bubbles were formed, nearly three feet in diameter (Boudet, 1876). Scum covered the surface at times for a distance of two miles below the sewer outlets. Sludge banks ten feet deep were formed at points of sluggish flow. The analyses below show the progressive changes in the character of the water.

TABLE XI

DISSOLVED OXYGEN IN THE SEINE AT PARIS

August–October, 1874. Boudet, 1876.

Station.	Distance below Paris. Miles.	Dissolved oxygen, c.c. per liter.
Pont d'Ivry	4*	9.50
Pont de la Tournelle	0	8.05
Viaduc d'Auteuil	5	5.99
Pont de Billancourt	6	5.69
Pont de Sèvres	7.5	5.40
Barrage de Suresnes	10.5	5.32
Pont d'Asnières	14.5	5.34
Pont de Clichy	15.5	4.60
Pont de Saint-Ouen	16	4.07
Pont de Saint-Denis	17.5	2.65
La Briche	19	1.02
Epinay	19.5	1.05
Pont d'Argenteuil	20.5	1.45
Barrage de Bezous	25	1.54
Ponts de Chaton	28	1.61
Écluses de Bougival	30.5	1.91
Ponts de Maisons	36	3.74
Pont de Poissy	49	6.12
Pont de Triel	53	7.07
Pont de Meulan	58	8.17
Mantes	68	8.96
Vernon	94	10.40

* Above Paris.

At a short distance above the main city the pollution began, and continued on a small scale, from minor drains and sewers, to the Pont d'Asnières. Below this point one of the main interceptors and below Pont de Saint-Denis a second, the

Collecteur du Nord, were discharged. These successive additions of organic matter reduced the oxygen present to one-ninth of its original amount. For a distance of over ten miles, from Epinay to Écluses de Bougival, the restoration of oxygen was only slightly in excess of its consumption. After this point, however, an improvement began, accelerated by the entrance of the purer River Oise below Pont de Maisons.

An interesting change in fauna and flora accompanies the gradual purification of a stream and the approximate completion of the task is made obvious, even to the eye, by the characteristics of plant growth. At points of extreme pollution no green growth appears, but only the molds and other colorless plants which require organic matter for their food. A grayish or blackish slimy mass of *Leptomitus* covers the rocks. Masses of *Beggiatoa* may float on the surface, and microscopic examination shows only a few Diatoms and Protozoa with occasional filaments of Blue-green Algæ. As purification progresses the mold-fungi disappear and the true green Algæ take their place, thriving on the nitrates formed from the decomposition of organic compounds. *Spirogyra* and *Conferva* and the desmids produce abundant rich green growths. Diatoms are present in greater variety and the *Mastigophera* take the place of ciliated Protozoa.

Practical Limits of Purifying Capacity. It is obviously of great importance to determine how much sewage a given stream will consume without danger of passing the critical point. This problem is a fairly complex one, since the capacity of a stream at any point must vary with its composition and the composition of the sewage, as well as with their relative flow. Rideal (1906) has attempted to express the relation between the various factors involved in the form of an equation, $XO = C\,(M - N)\,S$, where X = flow of a stream, O = parts of dissolved oxygen in the water of the stream per unit flow; S = volume of sewage or effluent; M = parts of oxygen consumed by a unit volume of sewage; N = parts of available oxygen in the form of nitrites and nitrates, and C = a constant.

Certain rules of a simple character may be laid down for practical purposes as a result of numerous studies of various rivers, and on the assumption of a fair normal composition for stream and sewage. Stearns (1890) gave as his opinion that if the flow of a stream is less than 2 cu. ft. per second for each one thousand persons connected with the sewers flowing into the stream, the amount of pollution is inadmissible. If the flow is greater than 8 cu. ft. per second for each one thousand persons connected with the sewers, the amount of pollution in the stream will not cause offence. Between the two limits is debatable ground. Rudolph Hering (1888), drawing his conclusions from the work of the Mass. State Board of Health, states that if the flow is less than $2\frac{1}{2}$ cu. ft. per second per thousand persons (or one gallon per minute per person) an offense is almost sure to arise, but when it exceeds 7 cu. ft. per second per thousand persons, safety is assured. Mr. Goodnough, in a Report to the Committee on the Charles River Dam, 1903, states: "Omitting reference to objections caused by the manner of discharge of sewage and objections which may be due to various other circumstances, and considering only the question as to whether objectionable conditions exist in the various streams into which sewage is discharged by reason of the quantity of sewage discharged, an examination of all the information available from the investigations that have been made shows that where the flow of a stream exceeds 6 cubic feet per second per 1000 persons discharging sewage, objectionable conditions are unlikely to result."

The statement of Mr. Stearns, Mr. Hering and Mr. Goodnough had regard only to the decomposition of organic matter causing offense and did not contemplate the use of water for manufacturing purposes, or the use of a stream for a water supply; with our present knowledge it cannot be stated that any degree of dilution will make the water entirely safe for the latter purpose.

It should also be noted that Mr. Stearns in giving his opinion regarding pollution says: " My conclusions relate to the pollution

of the water itself, as if the sewage was emptied into a stream of unvarying volume, flowing with sufficient rapidity to prevent deposits. If, instead, the sewage is turned into a stream where it is ponded by a dam, or if there are ponds on the stream below the point of discharge, the solid particles of the sewage may accumulate and decompose, giving off offensive gases. This is more likely to occur if the deposits are covered with foul water in which the dissolved oxygen has been used up, because the decomposition will then be putrefactive rather than a process of oxidation. The fluctuations in the height of a stream, where they cause large areas to be alternately covered with water and left bare, are also unfavorable for the proper disposal of sewage. In short, there are many things, such as the variations in the volume flowing in a stream occasioned by its use for mill purposes, the amount and character of manufacturing wastes, and the subsequent use of the water for different kinds of manufacturing, which require careful consideration in each case, and often a considerable variation from any general rules which may be laid down."

The statements above may be converted into figures which indicate the analytical condition of the stream after it has received its quota of sewage by a table given by Stearns (1890) in the article cited above. Assuming sewage and stream to be of average composition, a dilution of 2.0 second-feet per 1000 persons would mean 1.4 parts per million of free ammonia in the stream. Goodnough's danger line of 3.5 second-feet would give .8 parts per million and his safe point of 6.0 second-feet would be about .5 parts. Stearns's safe limit of 8.0 second-feet would give .4 parts.

A still more useful method of statement involves the relative volume of sewage and stream flow, assuming otherwise average conditions. Johnson (1905) has converted Hering's and Goodnough's figures into dilution volumes as follows:

TABLE XII

PROPORTIONS OF SEWAGE WHICH CAN BE DISCHARGED INTO A STREAM WITH SAFETY

(Johnson, 1905.)

Authority.	Nuisance probable.	Nuisance improbable.
Hering	1 in 16	1 in 45
Goodnough	1 in 23	1 in 36

Roughly, then, it may be said that a stream will purify one-fiftieth of its volume of sewage, but not one-twentieth. With a very sluggish stream sludge banks may accumulate and cause local nuisance with a dilution as high as one hundred parts or more of water to one of sewage (as in the Spree at Berlin). In summer the danger of putrefaction is much greater than at other times. Stream flow is at a minimum, while high temperature makes bacterial decomposition rapid and the need for oxygen correspondingly immediate. Thus a stream which is in good condition for most of the time may be dangerously near the critical point in the late summer. The table and diagram on page 39 (Winslow and Phelps, 1906) illustrate this condition as exemplified in the Merrimac River. The flow of this stream is always above Goodnough's limit of six second-feet, but it approaches it closely in September and October. The coincident high temperature leads to a rapid decrease of dissolved oxygen. The importance of the temperature factor is strikingly shown by the curve for November and December, 1899; with no increase in dilution a fall in temperature, with its consequent slackening of fermentation processes, shows a marked rise in dissolved oxygen. Although at the lowest points the dissolved oxygen averages do not show complete exhaustion, the river is sometimes distinctly offensive during the summer. Theoretically, while any dissolved oxygen remains there should not be putrefaction; practically, any value below 50 per cent

of saturation is likely to be accompanied at times by malodorous conditions.

FIG. 7. Relation of Dissolved Oxygen to Flow and Temperature in the Merrimac River.

TABLE XIII

SEASONAL CONDITIONS IN MERRIMAC RIVER AT LAWRENCE, MASS.

(Winslow and Phelps, 1906.)

Month.	1899.			1900.		
	Flow per 1000 persons discharging sewage (second-feet).	Temperature, deg. F.	Dissolved oxygen (per cent of saturation).	Flow per 1000 persons discharging sewage (second-feet).	Temperature, deg. F.	Dissolved oxygen (per cent of saturation).
January	42.6	34	96.3	18.2	33	81.6
February	26.4	34	88.1	89.1	34	87.8
March	64.5	34	95.6	87.7	35	....
April	143.2	......	99.3	100.0	41	99.1
May	51.6	58	84.4	54.1	54	....
June	16.1	73	71.1	21.4	73	62.1
July	13.4	76	66.6	9.8	77	59.4
August	11.3	74	58.3	10.1	75	43.6
September	10.8	67	57.2	8.2	71	32.5
October	9.7	58	53.7	13.6	62	47.6
November	15.1	40	78.1	31.6	46	91.2
December	15.1	36	84.3	36.6	38	98.0
Average	34.5	53	77.8	39.9	53	70.3

Legal Control of Stream Pollution. It is evident that the discharge of unpurified sewage into the nearest stream may often offer an eminently satisfactory method of disposal as far as the polluting community itself is concerned. In such cases it is the other communities below which suffer; and it is important to consider how responsibility can be brought home to the offending party. The first recourse is naturally to the common law. The principle is well-established that every riparian owner has a right to the reasonable use of the water of the stream and has a right to it in a natural state and unpolluted, except for the reasonable use of other riparian owners. The same general law applies to ponds and lakes, unless specifically modified by statute (Choate, 1908). The vital point of this legal principle in practice lies in the definition of "reasonable use." When the common law grew up, reasonable use meant use for washing, watering cattle, irrigating land, bathing, fishing and other purposes incident to agricultural life. All these things are still of course reasonable uses and for whatever detriment they work there is no redress.

With the growth of manufacturing, however, new problems have arisen which are much more complicated. If the farmer is entitled to use a stream for all his ordinary needs, why has not the manufacturer the same right to use the stream for his needs, which may be primarily the disposal of industrial wastes? Because, as the courts appear to hold, the agricultural use does not seriously affect the character of a stream, while the industrial use may, and often does, affect it very fundamentally. It is true that in Pennsylvania the courts have decided the discharge of water pumped out from the coal mines and laden with acid wastes to be a proper use, in spite of the fact that the water of the stream was rendered practically useless for domestic purposes. The general trend of opinion is, however, in the other direction. The Supreme Court of Massachusetts in the case of Parker against the American Woolen Company rendered a sweeping decision in the following terms: "The plaintiff will restrain the defendant from discharging into the stream

any noxious or offensive substances to such an amount or in such a quantity as to affect noticeably or appreciably the purity of the water, when it reaches the plaintiff's premises, so as to render it materially less fit for drinking or for other uses than it was when it entered the defendant's premises."

Whatever the ultimate position of the courts may be in regard to industrial wastes, the principle is well established that the discharge of domestic sewage in such amounts as substantially and appreciably to alter the natural character of a stream is an injury for which both damages and injunctions may be obtained by riparian owners.

The construction of the largest new sewage disposal plant in the United States, at Columbus, Ohio, was undertaken only under the pressure of litigation, in the course of which the city was directed to pay heavy damages to riparian owners along Alum Creek. The courts of New Jersey have taken an advanced stand on this subject in several recent cases. It has even been affirmed in one instance that surface drainage, when carried in artificial pipes rather than over the surface of the ground, became by that fact polluting material and might not be discharged into a watercourse unpurified.

Disposal of Sewage in Tidal Waters. Cities situated on the seaboard are fortunate in having the most favorable conditions for disposal by dilution. The volume of diluting water is large and the pollution of water supplies is not to be feared. The commonest procedure is a direct discharge by numerous small sewers into tidewater.

In the case of New York City the question of harbor pollution has been carefully studied by several Commissions (Soper, 1906); and it appears that some 450,000,000 gallons of sewage are daily discharged into the harbor. The water shows evidences of pollution (.1 part per million of free ammonia), and analyses are not much better on the incoming than on the outgoing tide, showing that much of the polluting material is washed back and forth instead of being flushed out to sea. The harbor takes care of all that is discharged into it, without creating any general

nuisance. At particular points, however, conditions are very bad, as in Newtown Creek, a tributary of the East River, and in the Gowanus Canal in Brooklyn; and the Metropolitan Sewerage Commission in its last report states that the harbor in general "is more polluted than considerations of public health and welfare should allow."

Local nuisances have led in many seaport towns to the discontinuance of discharge from pierheads and the substitution of intercepting and outfall sewers discharging at some distance out to sea. Boston offers a good example of this latter method. Since 1895 two main sewers have discharged into the harbor, serving the city and surrounding metropolitan district, which includes 25 cities and towns, with a territory of nearly 20 square miles. The sewage of the region north of the Charles flows continuously from an outlet near Deer Island Light and averaged 59,800,000 gallons per day in 1908. The sewage from the region south of the Charles has been discharged since 1884 at Moon Island, nearer the center of the harbor. Here, in order to protect the adjacent shores, it has been thought necessary to hold the sewage in four masonry basins and to discharge it only on the outgoing tide. An average of 87,661,058 gallons a day passed out at the outlet in 1908. On September 19, 1904, a third outlet was opened to take the sewage from certain high-level regions in the south metropolitan district. This discharges continuously in the outer harbor near Nut Island and delivered 37,800,000 gallons per day in 1908. Experience has shown that no serious nuisance is caused by the Deer Island and Nut Island outlets. The sewage at Deer Island disappears within $1\frac{1}{4}$ miles of the outlet, while off Moon Island the sewage stream may be traced outward round the south end of Long Island for perhaps two miles. In both cases passing boats find the immediate vicinity of the outlet unpleasant, and near Moon Island the value of property on the mainland is said to be affected. No serious menace to health, however, is involved and the sewage apparently produces no permanent damage in the harbor. So popular is this method of disposal in the sea that according to

a review made by the Massachusetts State Board of Health in 1902 (Massachusetts, 1903) nearly one-half the population of that State was tributary to such systems. In general they have proved successful, although a serious nuisance is created in some places, as at Lynn, where the sewage is discharged in shallow water and over tidal flats. It is certain that such methods of disposal will prove less and less satisfactory from year to year as the volume of sewage and the concentration of shore population increase.

Where tidal waters are used for the cultivation or storage of shellfish the problem takes on a new aspect. Direct discharge of unpurified sewage is here undesirable because the disease germs present menace the purity of an important food supply. This question has been exhaustively treated by Fuller (1905 *b*), who shows that the annual value of the shellfish taken along the Atlantic and Gulf coasts is over fifteen million dollars. Nearly half of this crop is grown in the waters of the Delaware and Chesapeake Bays, into which the sewage of Philadelphia, Wilmington, Baltimore and Washington is discharged. In England, too, this problem has attracted wide attention. The Royal Sewage Commission in an extensive report concluded that the consumption of polluted shellfish was a grave evil, but that it must be met, less by restricting sewage disposal than by regulating the taking and storing of shellfish (R. S. C., 1904 *a*). With such an industry as that of Chesapeake Bay this method of procedure will scarcely suffice. When the city of Baltimore undertook its present study of the sewage problem the Sewerage Commission required that " the effluent proposed to be discharged into the Chesapeake Bay or its tributaries in the system to be recommended by the engineers shall be of the highest practicable degree of purity." In pursuance of this requirement and as a concession to the demand for a bacterially purified effluent which should not menace oyster beds, the Commission included in its preliminary estimates over a million dollars for supplementary sand filters in addition to the trickling filter plant required for organic stability (Baltimore, 1906).

A very curious problem has arisen at Belfast, Ireland, in connection with the discharge of sewage into tidal waters. Here neither decomposing organic matter *per se* nor disease germs are the cause for complaint. The trouble has arisen from immense growths of the green sea-lettuce, *Ulva*. This alga thrives not only on free ammonia, but, like other green plants, on nitrates as well, and complete oxidation would leave the sewage nitrogen still in a form to favor its development. Professor Letts (1908) has devised a special process of purification involving nitrification and subsequent denitrification to meet this need; and with such a combination he is able to get rid of a considerable amount of the nitrogen present by liberating it in the gaseous form.

The General Field for Disposal by Dilution. It is clear from what has been said that disposal of sewage by discharge into lakes, streams and tidal waters is a real method of purification. Under certain circumstances it is the economical and proper one. Thus the plans for sewage disposal at New Orleans provide for discharge, after rough preliminary screening, into the Mississippi River, at a point well beneath the surface and out from the shore. There are no shellfish layings and no water supplies below; and the vast volume of the river is amply sufficient to absorb the polluting material without the slightest danger of offensive conditions.

In Germany, where the rivers are large, if not of Mississippi dimensions, disposal by dilution has been extensively adopted under rational and scientific conditions, careful screening being generally a preliminary requisite. Thus at Cologne the sewage is passed through a fine screen, and thence through a small sedimentation basin to the Rhine River. It is provided that disinfection shall be carried out in the basin upon any occasion when the Imperial Board of Health requires it. A similar treatment is proposed in connection with the Passaic trunk sewer of New Jersey, which is to discharge the sewage of the cities and towns in that valley into New York harbor; and this method may often be allowable for cities and towns situated on large rivers and on the seacoast, but not for inland cities situated on small streams.

CHAPTER III

SCREENING AND STRAINING OF SEWAGE

Objects of Screening. Screens were used originally in connection with sewage pumping plants for the purpose of removing from the sewage such materials as would tend to clog and possibly break the pumps. They were also used to prevent stoppage in sewers where surface water is admitted, or at the upper ends of inverted siphons.

Where raw sewage is applied directly to land or artificial filters, whether the filtering material be fine or coarse, it is almost always economical to screen the sewage in order to prevent rapid clogging of the filtering surface and loss in capacity as well as the consequent production of a poor effluent. Where the sewage first passes into properly baffled tanks, before being applied to filters, screens are less essential, although generally desirable. Where sedimentation tanks are operated on the septic plan, however, it is particularly important to screen the entering sewage in order to remove the lighter materials which mass together and form an undue proportion of scum.

The rapidity of clogging, and hence the labor necessary for the care of a screen, depends, for a sewage containing a given amount of suspended matter, upon the size of open space in the screen and upon the area of screening surface. The usual practice in this country is to use screens having an open space of one-half inch or more, which may be kept clean without much labor or expense. In this case the removal of only the coarser suspended matter (paper, rags, garbage, corks, sticks, leaves, etc.) can be expected.

Modern foreign practice seems to be developing in the direction of more thorough screening than is generally the case in this country. In many German cities sewage is discharged into

the rivers with no purification other than that effected by fine screens; and the use of this method of disposal has led to the design of mechanical and automatically-cleaning screens of numerous types. As a result of extensive experiments at Cologne the authorities withdrew a stringent series of requirements for sewage purification and permitted the discharge of the Cologne sewage into the Rhine after the removal of suspended matter by screening to a diameter of one-eighth inch. The recent agreement between the Passaic Valley Sewerage Commission and the United States government as to the discharge of the sewage from the new Passaic Valley sewer similarly calls for screens having an opening of not over .4 inch followed by sedimentation; and under these conditions the opposition of the National and State authorities has been withdrawn.

The actual percentage of suspended matter removed by ordinary screening apparatus is difficult to determine accurately because representative samples of unscreened sewage cannot readily be obtained. The removal is, however, comparatively small, ranging from 2 to 10 per cent with bar screens. With more complicated apparatus, where wire cloth is used, the per cent of organic matter removed may be as high as 25. From an economic standpoint, the question of operating these finer screens at comparatively great expense should be given careful consideration as balanced against the cost of removing the finely suspended matter by other methods.

There are three general classes of sewage screens: first, hand operated screens, or gratings; second, mechanically operated screens, gratings or sieves; third, coarse filters, or strainers.

In the first class may be included bar screens or gratings, wire mesh screens, basket screens and upward flow screens; and in the second class may be included stationary screens with automatically moving rakes, cylindrical revolving screens equipped with combs or brushes, and other self-cleansing screens of various types.

Bar Screens or Gratings. The most satisfactory type of screen for use at small plants and under conditions where only

the coarsest suspended matter is to be removed, consists of a set of parallel iron bars or rods placed on end and spaced so as to provide openings of the desired width. Wooden screens, similarly designed, have been used. The plane occupied by the screen thus formed should be inclined downstream and should make an angle of about 30 degrees with the vertical (see Fig. 19).

Bar screens may readily be cleaned by means of hand rakes, the teeth of which fit into the spaces between the bars or rods. If it is necessary to place the screen much below the ground level, it can be cleaned by a long-handled rake; or the screen itself may be lifted to the surface for cleaning by means of pulleys operated by hand or power.

The usual open space for the bar type of screens is $\frac{1}{2}$ to 1 inch or more. Data for a number of Massachusetts screening plants, cited in the table on page 59, give a fair indication of American practice. The opinion is growing, however, that a finer grating may be often both practicable and desirable. At Columbus, Ohio, there are three sets of screens, the first of 3 by $\frac{3}{4}$-inch bars set 6 inches in the clear, the second of $\frac{3}{4}$-inch rods 1 inch in the clear, and the third of $\frac{3}{8}$-inch rods, $\frac{1}{2}$ inch in the clear. At Washington, Pa., there are two sets of bar screens 4 feet by 3 feet outside dimensions, the first set having an open space of $\frac{3}{8}$ inch and the second set a space of $\frac{1}{4}$ inch. These screens need cleaning every hour or so during the day when the sewage is strongest, but only once in three or four hours during the night. It has not been necessary to employ any additional attendant to do this work, however, and the advantage derived from using screens with so small an open space is very marked.

Wire Mesh Screens. Wire screens composed of heavy wire netting (about $\frac{1}{8}$ inch in diameter) having a rectangular mesh of $\frac{1}{2}$ inch to 1 inch are the cheapest type of screen. This feature affords the chief excuse for their use, as they are open to the serious objection of being very difficult to clean. At Marion, Ohio, there is an example of the unsuccessful use of this type of screen (Ohio, 1908). At the Columbus Sewage Testing Station

wire mesh screens having $\frac{1}{2}$ inch to $\frac{3}{8}$ inch openings were used during the experiments, but required frequent attention (Johnson, 1905).

Basket Screens. Where the flow of sewage falls in a vertical stream it is possible to use basket screens or cages at the top. These screens are made either of woven wire with rectangular mesh or of bars. Flat horizontal screens, placed to receive a vertical current of sewage, might be included in cases where the sewage is discharged into a tank or receiving reservoir at an elevation above the high-water level. Cleaning may be accomplished by long-handled rakes or by hoisting the basket to the surface of the ground. If a basket screen is placed under the inlet to a reservoir at such elevation that it is at times partially submerged, it ceases to act as a basket screen, strictly speaking, and becomes a bar screen or wire mesh screen, according to its construction. At Bonn the municipal authorities in 1903 ordered basket screens to be placed in the manholes at junctions of all lateral sewers with the interceptor. These screens, which are raised to the surface for cleaning, are intended to remove all particles greater than $\frac{1}{8}$ inch. Basket screens are objectionable for the reason that the force of the sewage dropping vertically tends to disintegrate the screenings and cause them to pass through the screen. This type should, therefore, be used only where the flow of sewage is very small.

Upward Flow Screens. Flat screens, of bar construction or having a rectangular mesh, are sometimes placed in a horizontal position to receive an upward current of sewage. The sewage is first conducted to the *bottom* of a small well or sump, from which it rises and passes through the horizontal screen which occupies the entire cross section of the sump. There is an example of this arrangement at the State Reformatory at Mansfield, Ohio (Ohio, 1908). The serious objection to this method is that the under side of the screens are not readily accessible for cleaning. Although the sump beneath the screen may be freed from deposits by draining and flushing, the screenings always tend to cling to the screen.

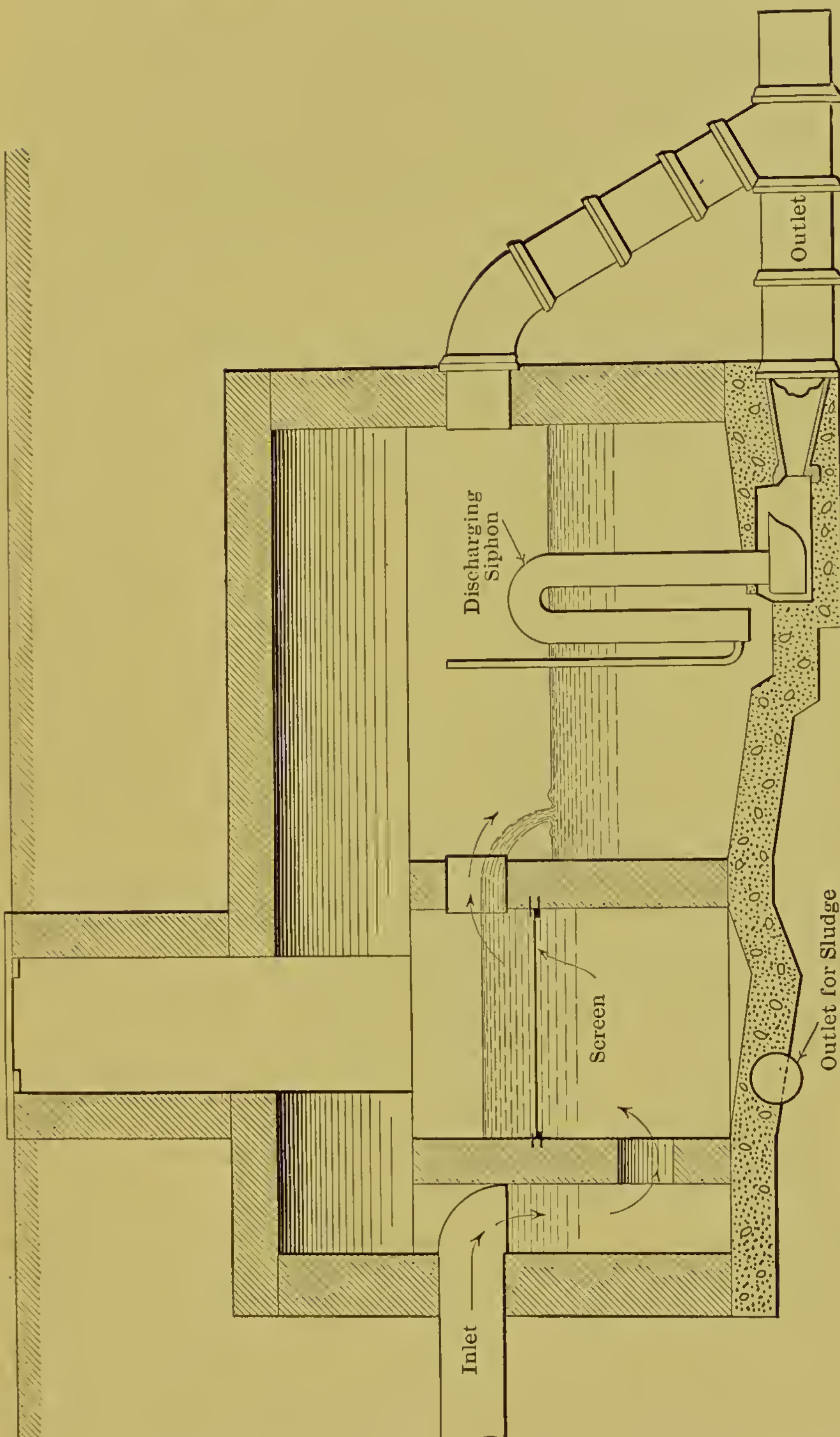


FIG. 8. Screen Chamber for Upward Flow Screen.

Mechanically Operated Screens. With an increasing recognition of the importance of screening out the coarser suspended matter from sewage, much thought has been given to the problem of designing screens for large plants, where a high degree of clarification is desirable and where hand cleaning is not

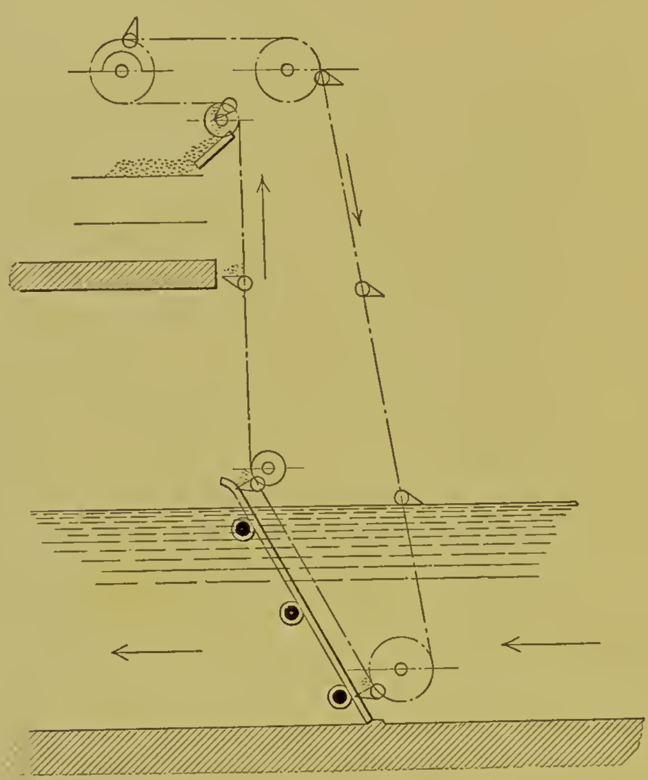


FIG. 9. Stationary Screen with Power-driven Rakes. (Copied by permission from Dunbar, 1908.)

satisfactory. As a result, mechanical or power-operated screens — also called "self-cleansing" and "automatic" screens — are much used in Germany and England and are now being introduced in the United States.

The greatest advances in the art of mechanical screening have been made in Germany, where screens having openings as small as $\frac{1}{25}$ of an inch are used. The screening plants usually

have two, three or more screens, of different-sized openings, through which the sewage passes successively. The screens are cleaned in various ways, by scrapers, brushes, rakes, combs, water jets and compressed air. The screenings are usually removed by means of belt conveyers to tramcars. The problem of keeping these fine screens from clogging, by automatic means, is a complex one, as the tools or appliances used for cleaning the screens must themselves be kept clean by additional apparatus. It is therefore not surprising to find a large variety of designs, none of which are perfect.

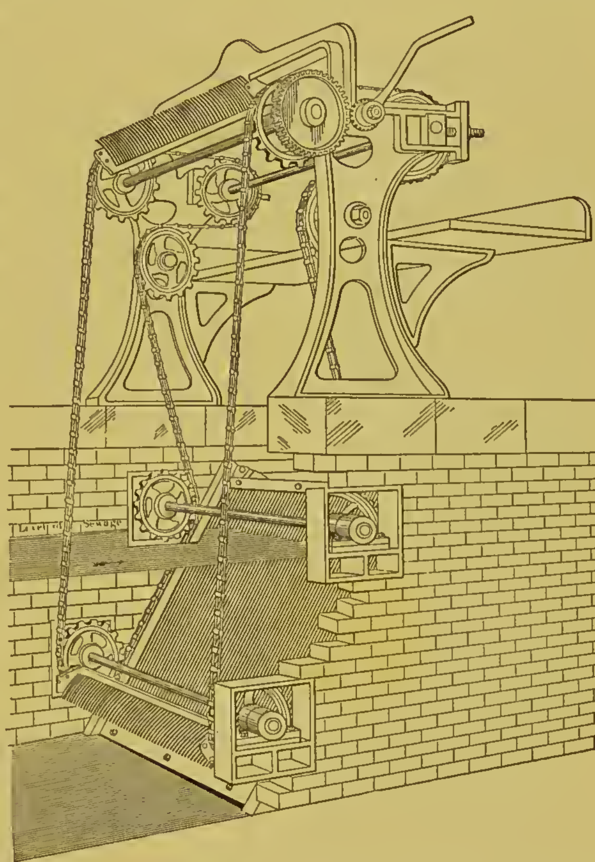


FIG. 10. Sewage Screen Cleaned by Mechanical Rakes. (Copied by permission from Raikes, 1908.)

Stationary Screens with Power-Driven Rakes or Brushes. A common type of mechanical screen consists of fixed sets of parallel bars or slats, between which are continuously moving a series of power-driven rakes. The teeth of these rakes are so spaced as to exactly fit into the spaces between the slats. The rakes are attached to two endless chains, one at either end of the screen, and by a suitable arrangement of the pulley wheels which guide these chains, the rakes may be deflected downward on reaching the top of the screen so that the material collected

falls onto a platform, from which it is removed, either automatically or by hand, (Fig. 9.) Raikes (1908) gives an excellent description of a mechanical screen of the bar type manufactured by Messrs. S. S. Stott & Co. of Haslingden, which is provided with special appliances for cleaning the rakes, (Fig. 10.)

In some cases, as at Bromberg and at Wiesbaden, the screens or sieves are placed horizontally, so that the sewage drops through them, and are cleaned by brushes attached to an endless chain.

Revolving Screens. There have been in operation for ten years or more, both in England and Germany, many interesting forms of mechanical screens in which the screen or sieve is itself submerged in and moved through the sewage. This form is apparently considered more efficient than the fixed screen type, and is much more easily adapted to automatic cleaning.

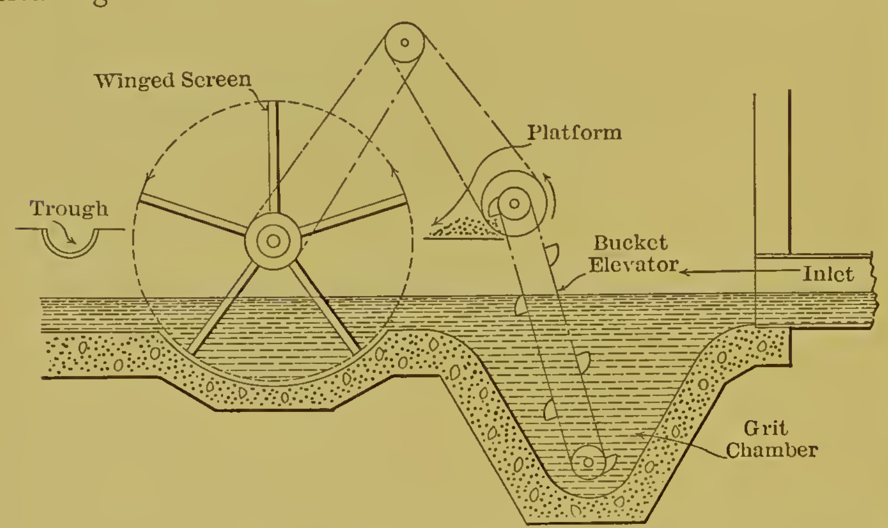


Fig. 11. Automatically Cleaned Grit Chamber and Winged Screen at Wiesbaden.

In Fig. 11 is illustrated the winged screen introduced in 1899 by Schneppendahl at Wiesbaden (Dunbar, 1908). Each of the five or six wings consists of a flat rigid bar or slat screen having an open space of ¼ inch or more. The bed of the sewer

or chamber beneath the screen is hollowed out in the form of a segment of a circle and the screen is revolved in a direction opposite to the flowing current of sewage. The accumulations are brushed off into a trough. This type of screen is usually preceded by a grit chamber as shown in the figure.

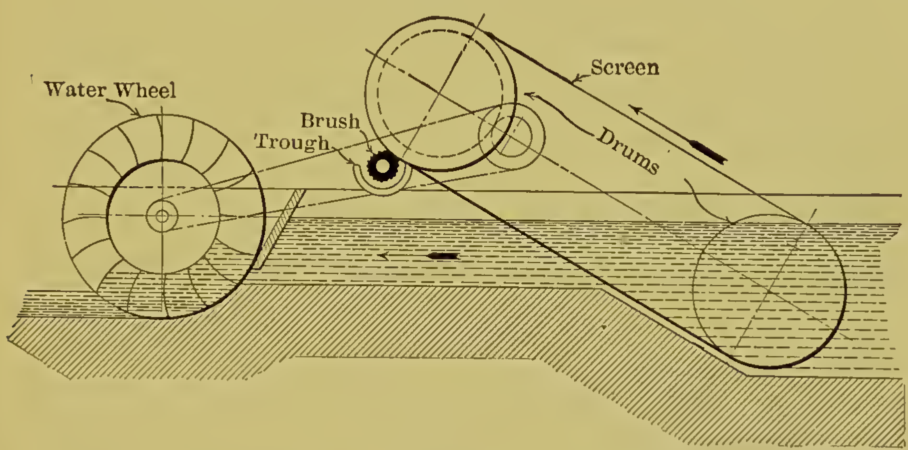


FIG. 12. Revolving Screen, Carshalton. (Copied by permission from Dunbar, 1908.)

Fig. 12 represents a form of screen manufactured by John Smith & Co., Carshalton, Eng. It consists of a woven wire screen or sieve, suspended from two revolving cylinders or drums, the lower one being immersed in the sewage transversely across the channel. The lower drum is sufficiently open in construction to allow the liquid to pass through. The intercepted material is caught upon the wire screen, lifted out of the sewage and over the upper drum, where it is removed by a revolving brush and falls into a trough, from which it is taken away either by hand or by a worm conveyer. The power to move the screen is generally derived from a water wheel driven by the current of sewage. The screen of this type at Göttingen travels at the rate of 2½ yards per minute. Good results have been obtained with a wire cloth sieve having an area of 1 per cent of that needed for a stationary screen, the linear velocity of the wire cloth being from 0.5 to 20 inches per

second, varying with the hourly fluctuations in flow (Kuichling, 1909).

FIG. 13. Revolving Screen at Birmingham, England. (Courtesy of J. D. Watson.)

Screens of the general design above described are in use at Hamburg and Glasgow. At these places, however, there are used, instead of woven wire, flat iron bars spaced about $\frac{3}{4}$ inch apart with angle irons so arranged as to prevent the intercepted material from sliding down.

Probably the most complicated mechanical screen is that shown in Fig. 14, designed by Riensch after ten years' study of all kinds of mechanical screens at Wiesbaden and Marburg. With this form a disc sieve is revolved in the sewage in a position nearly, but not exactly horizontal. The screenings are removed through a lateral trough by means of revolving brushes. The advantage claimed for this type is that the intercepted matter is not so likely to be crushed and forced through the openings, as is the case with other forms. The openings in the disc are made to suit the local sewage and may be as small as $\frac{1}{12}$ inch.

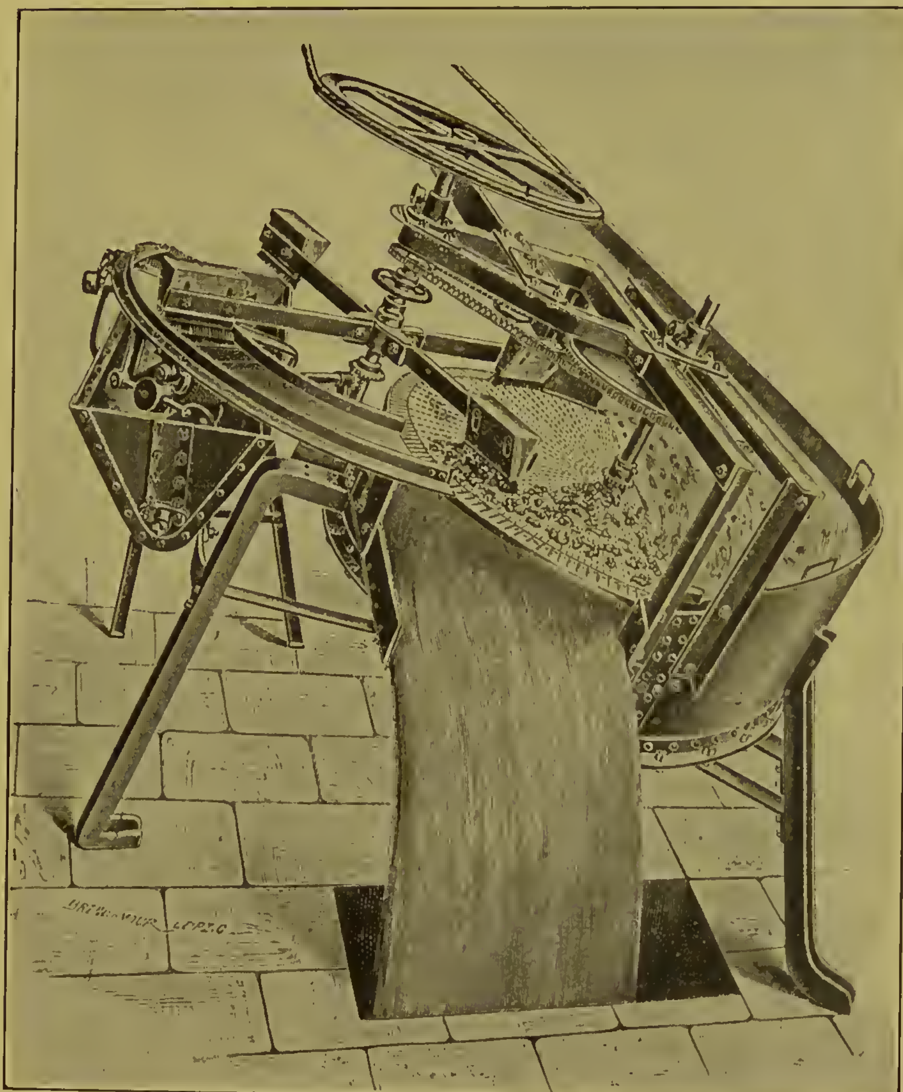


FIG. 14. Riensch's Disc Screen.

The most interesting revolving screen in the United States was designed and installed at Reading, Pa., by Mr. O. M. Weand (Hering and Fuller, consulting engineers for the plant). This screen, called by the designer a "segregator," consists in its improved form of a cylindrical iron framework about 6 feet in diameter and 16 feet long. Securely clamped to this framework is a brass wire (No. 12 gage) screen having a $\frac{5}{8}$-inch mesh.

This serves as a support to the brass wire cloth, having 40 meshes to the inch, which constitutes the screening surface. The latter is installed in segments which can be readily replaced as needed.

The apparatus shown in Figs. 15–17 is placed in a pit adjacent to the engine room of the station where the sewage is pumped to the purification works. The sewage from the main

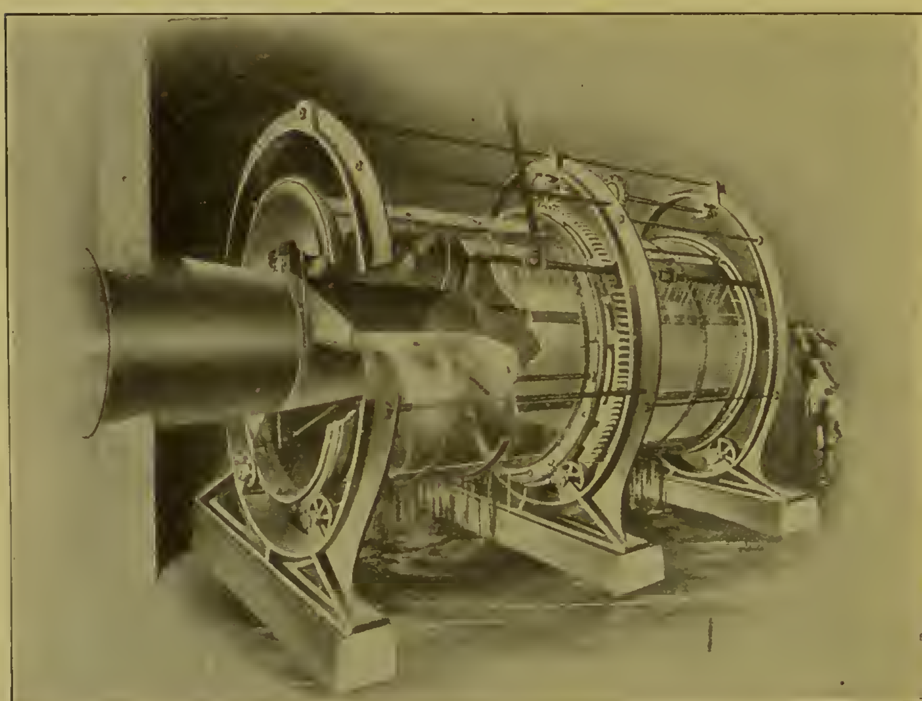


FIG. 15. General View of Reading Screen.

sewer is discharged through a 24-inch pipe into the barrel of the screen and passes outward through the wire cloth to the bottom of the pit and thence to the pumps. The segregator is revolved continuously at a rate of about six revolutions per minute, by means of power derived from one of the engines, transmitted through a line shaft, chain and sprocket to the driving shaft and thence through spur wheels, engaging gear rings which are rigidly attached to either end of the cylindrical framework.

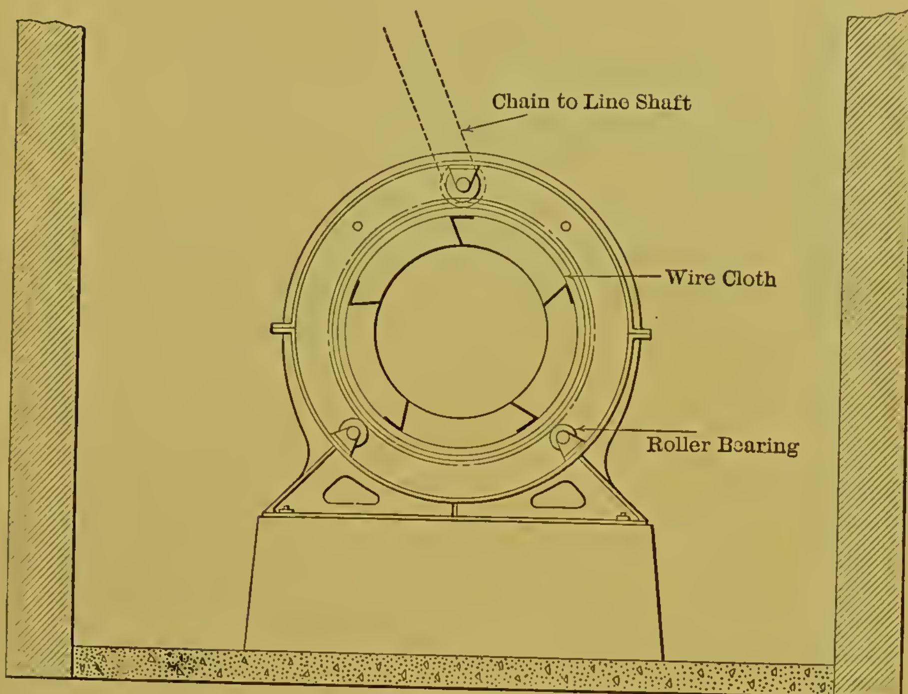


FIG. 16. Section of Reading Screen.

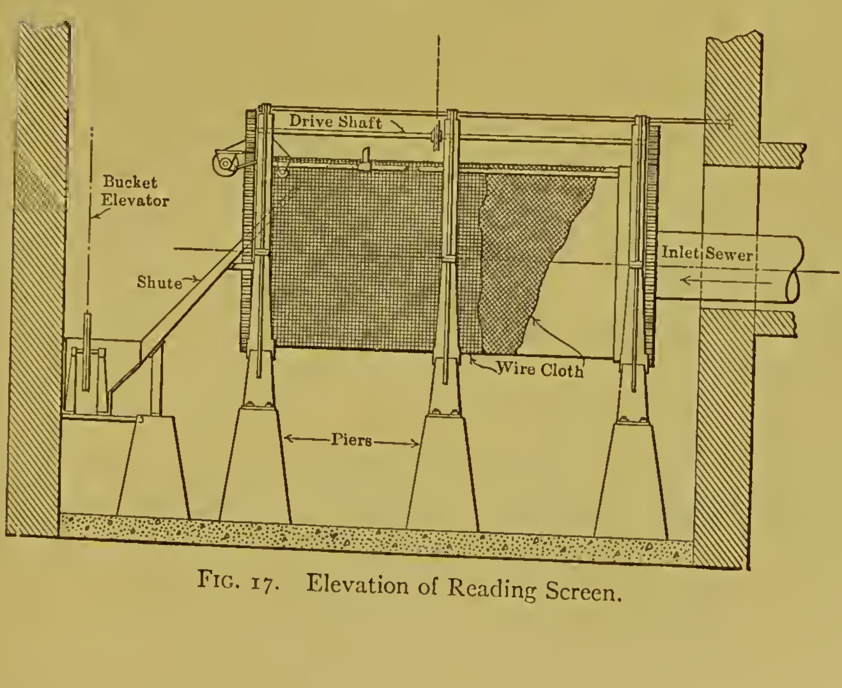


FIG. 17. Elevation of Reading Screen.

The accumulated screenings are removed from the wire cloth by means of jets of water directed against the outside of the screen. To the pipe supplying the water is given a longitudinal motion which covers the whole screen area by the water from the jets. This washing process causes the screenings to fall to the lowest point of the interior of the cylinder, where they are continuously pushed forward by a worm conveyer to the lower end of the screen. Rotating scoop buckets then deliver the screenings through a shute to a platform in the bottom of the pit. Thence this material is elevated by a belt conveyor to the operating floor above, where it is placed in bags, dried in a centrifugal dryer, mixed with coal and burned under the boilers. The screenings, before drying, range from about 20 to 30 cubic feet per million gallons of sewage and contain about 90 per cent of water, which is reduced to 75 per cent by the centrifugal process.

Design of Screen Chambers and Required Screening Area. The proper design of screen chambers has an important bearing upon the success of the screening process. Screen chambers should be placed as near as possible to the surface of the ground in order to facilitate handling the screenings. They should furthermore be located near a pumping station or near the headquarters of the man in charge of the plant, in order to facilitate inspection and cleaning. They should be well ventilated, and the screens themselves should be placed at such an elevation that no serious damage would result if they should accidentally become so clogged as to obstruct the flow of sewage. Screen chambers sometimes serve also as grit chambers or detritus tanks, in which case the sludge must be regularly removed.

The design of a screen chamber is intimately related to the area of screen used, and this in turn depends upon the character and amount of sewage to be treated, the size of the screen, and especially upon the amount of attention which is to be given to keeping the screen clean. Speaking generally, for the ordinary bar or slat screen, the total open space should be somewhat greater than the cross section of the main sewer. Ogden (1908)

advises making the free area 50 per cent greater than the cross section of the sewer, which would usually mean making the total screen area about 300 per cent greater than such cross section. Raikes (1908) states that a width of 9 inches per 1000 population is the usual basis of calculation for large works; although this must depend upon the nature of the sewage, fineness of screens and maximum flow.

The following table shows the sizes of screens used in various Massachusetts municipalities, the statistics referring in all cases to screens of the fixed bar type.

TABLE XIV

SCREEN CHAMBER DATA FOR MASSACHUSETTS TOWNS

(Mass. 1904.)

Town.	Average sewage flow, gal. per day.	Open space in screen (inches).	Screening area (sq. ft.)	Amount of screenings removed.
Andover	125,000	$\frac{1}{2}$	18	$\frac{1}{2}$ cu. ft. per week.
Brockton	878,000	$\frac{3}{4}$	100	185 lbs. per day.
Clinton	785,000	$\frac{1}{2}$	117.5	
Framingham	652,000	$\frac{3}{4}$	60	Wheelbarrow load twice a week.
Gardner *	250,000	$1-\frac{1}{2}$	3.88	
Natick	566,000	$\frac{3}{4}$	22	One bucketful in two weeks.
Pittsfield	1,456,000	1″ 1st screen; $\frac{3}{4}$″ 2d screen	30 ca.	2 wheelbarrow loads per day.
Southbridge	350,000	$\frac{3}{4}$	56	
Spencer	375,000	$\frac{3}{4}$	70	4 wheelbarrow loads daily.
Stockbridge	75,000	$\frac{3}{4}$″ 1st screen; $\frac{1}{2}$″ 2d screen	42, 1st; 8, 2d	2 wheelbarrow loads a day.
Westborough	282,000	$\frac{1}{2}$	20	

* Templeton System.

In Figures 18 and 19 are shown a general view and certain details of the screen chamber at Washington, Pa., which, fortunately, could be placed near the surface of the ground so that the screenings can be readily removed by means of a rake, placed in wheelbarrows and hauled to the area chosen for final disposal.

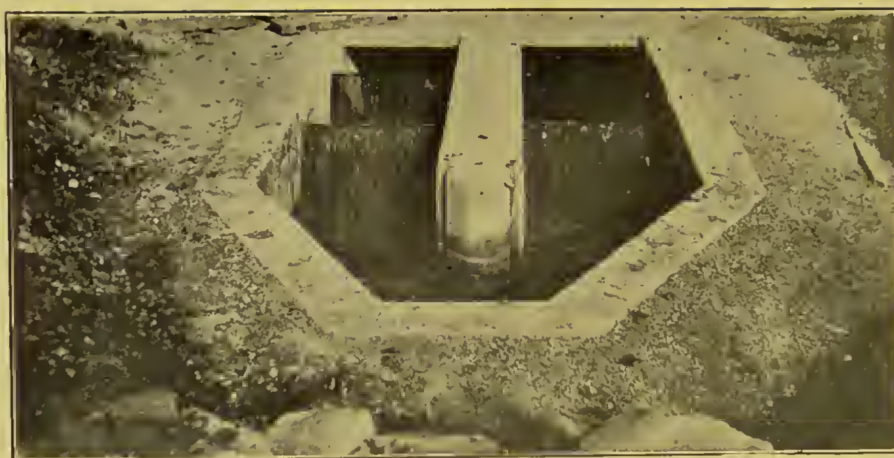


FIG. 18. View of Screen Chamber at Washington, Pa.

Efficiency of Screening. On account of the practical difficulties incidental to collecting truly representative samples of unscreened sewage and to determining, by analytical methods, the exact amount of suspended matter therein, it is difficult to discuss the efficiency of screens, in terms of per cent removal, as is done in connection with tanks or filters receiving screened sewage.

Such data as are available refer to the amount of solid screenings produced rather than to any comparison of screened and unscreened sewage. These data are expressed sometimes in units of weight and sometimes in units of volume, and the relation of these two units is highly variable. Kuichling (1909), in discussing the efficiency of certain mechanical screens, assumes that one cubic foot of wet screenings (83 per cent moisture) weighs 30 pounds. Bredtschneider (1905) assumes that a cubic foot of wet suspended matter weighs 23 pounds; and Monti found this volume (with 56 per cent moisture) to weigh 20 pounds.

Considerably higher estimates, based on experience in this country, are given by Johnson (1905), who reports that at the Columbus experiment station, the wet screenings weighed 65 pounds per cubic foot; and Mr. W. M. Brown, Chief Engineer

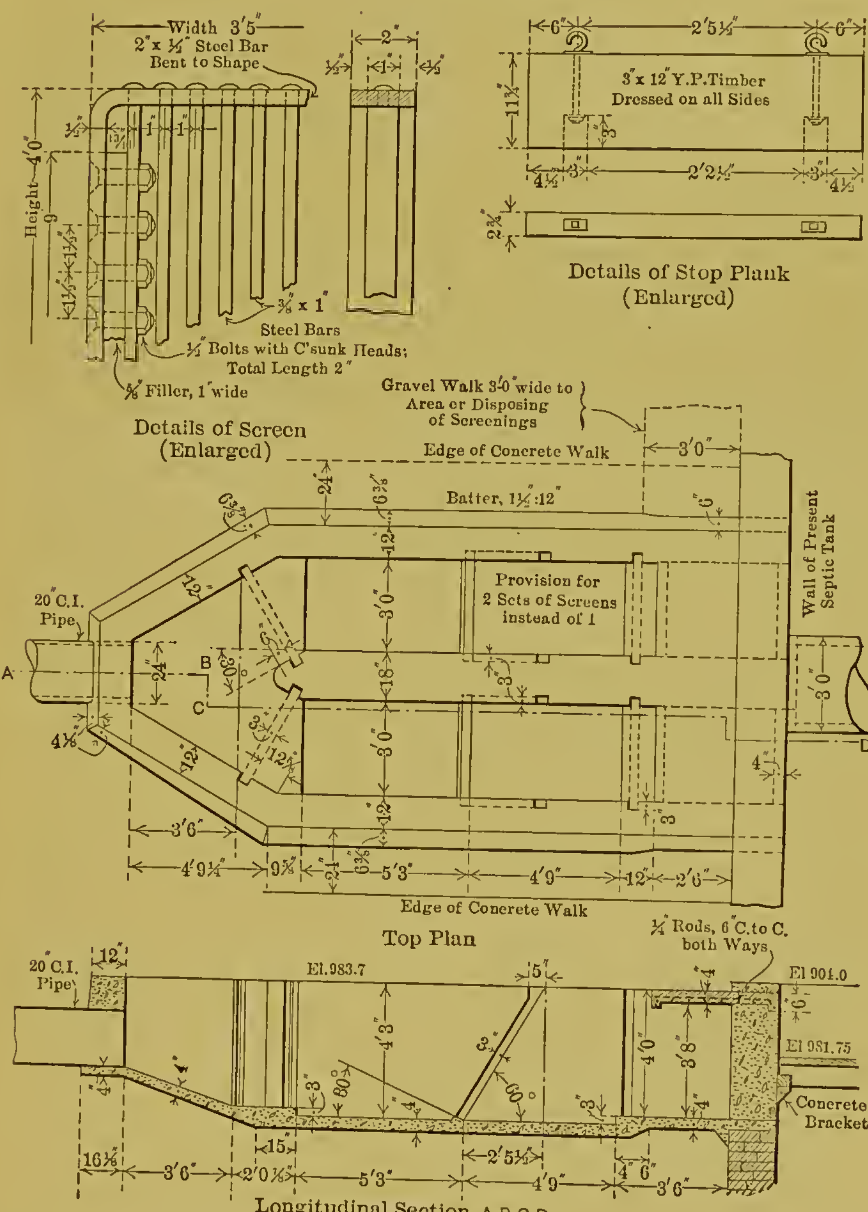


FIG. 19. Detail of Screen Chamber at Washington, Pa.

of the Metropolitan sewerage works at Boston, estimates the weight of wet screenings at 60 pounds.

The table below shows the results attained at the Boston Metropolitan screen chambers expressed in cubic feet and gives a fair idea of what can be done with fixed bar screens of large opening.

The screens of the North district are of $\frac{3}{4}$-inch bars set $1\frac{3}{4}$ inches center to center, giving an open space of 1 inch in the clear. At the pumping stations of the South district there are double screens, like the above, placed about an inch apart with the rods of each opposite the open space of the other. This gives an opening of almost $\frac{1}{8}$ inch in the direction of the sewage flow, while the diagonal opening is about an inch.

TABLE XV

AMOUNT OF SCREENINGS REMOVED IN THE METROPOLITAN SEWERAGE DISTRICTS OF MASSACHUSETTS

North District.			South District.		
Year.	Average daily quantity passing screens (million gals.)	Screenings removed (cu. ft. per million gallons sewage screened).	Year.	Average daily quantity passing screens (million gals.)	Screenings removed (cu. ft. per million gallons sewage screened).
1904	57.2	2.7	1905	25.0	5.9
1905	54.4	2.7	1906	33.6	6.5
1906	58.1	2.8	1907	40.6	5.0
1907	64.3	2.8	1908	37.8	4.3
1908	59.8	2.8	....	....	....

The figures tabulated on page 62 are given by Mr. E. H. Beard for the Reading plant, where the screen has 40 meshes to the inch.

As stated below, the fresh screenings at Reading contain 90 per cent of moisture and the dried screenings, only 75 per cent. Since ordinary bar screenings contain about 75 per cent of moisture the true effect of screening at Reading is to take out about four times as much solid material as is removed by the coarse gratings of the South Metropolitan district of Boston.

TABLE XVI

AMOUNT OF SCREENINGS REMOVED BY REVOLVING SCREEN AT READING, PA.

(Beard, 1909.)

Month.	Screenings dried by centrifugal machine. Screenings per million gallons.		Month.	Screenings dried by centrifugal machine. Screenings per million gallons.	
	Pounds.	Cubic feet.		Pounds.	Cubic feet.
February	810	....	August	....	15.66
March	570	....	September	....	30.24
April	930	....	October	....	18.90
May	1,020	....	November	....	19.17
June	1,300	....	December	....	18.36
July	1,390	....			

Further data for other screening plants are given in the table below.

TABLE XVII

EFFICIENCY OF SCREENING WITH SCREENS OF VARIOUS GRADES

(Johnson, 1905. Kuichling, 1909.)

Place.	Average daily quantity passing screens. Gallons.	Size of screen openings, clear.	Quantity removed per million gallons.	
			Cu. ft.	Pounds.
Providence, R. I.	19,500,000	1 inch	....	41
Boston, North	57,800,000	1 inch	2.8	168
Berlin	58,749,000	$\frac{1}{2}$ inch	10.6	212
Coventry	....	$\frac{2}{5}$ inch	....	250–300
Göttingen	....	$\frac{2}{5}$ inch	....	336
Columbus Testing Sta.	350,000	$\frac{1}{2}$ and $\frac{3}{8}$ inch	4.6	300
Boston, South	37,800,000	$\frac{1}{8}$–1 inch	4.3	258
Leeds	52,000	$\frac{1}{8}$ inch	5.5	....
Dresden	....	$\frac{1}{12}$ inch	23.1	....
Wiesbaden	....	$\frac{1}{2}$, $\frac{1}{5}$–$\frac{1}{8}$ and $\frac{1}{25}$ inch	36.9	....
Reading	2,040,000	$\frac{1}{50}$ inch	41.0	1,000

In general, it appears that screens with an open space in the neighborhood of $\frac{1}{2}$ inch or thereabouts will take out from 3 to 5 cubic feet of screenings per million gallons, while with clear openings of $\frac{1}{12}$ inch or less the material removed may rise to 20–40 cubic feet per million gallons. Kuichling (1910), assuming that wet screenings weigh 30 lbs. per cubic foot and contain 75 per cent

of moisture, calculates the removal in parts per million at the various places tabulated above as follows: Coventry, 7.5–9.0; Göttingen, 10.1; Reading, 90.0; Dresden, 177.8; Wiesbaden, 274.5.

Monti (1903) in a careful study of Berlin sewage passed through experimental screens of various openings from $\frac{7}{25}$ to $\frac{1}{50}$ inch found that the whole series took out between 10 and 14 per cent of the total dry suspended matter in the sewage. Of the total suspended matter removed by the five screens the $\frac{7}{25}$ inch screen took out 46 per cent, the three next screens together 24 per cent, and the $\frac{1}{50}$ inch mesh 30 per cent.

Cost of Screening. Little authentic information is available in regard to the cost of screening sewage. With small, and moderate-sized plants, the cost may be very slight, for the reason that no additional attendance is required over that necessary to run the plant as a whole. In other cases the cost of screening may form an appreciable item.

Below is stated, in tabular form, the cost of labor in operating screens at certain pumping stations in the Massachusetts Metropolitan Sewerage District for the year 1908. The large amount of sewage pumped at these stations makes necessary the employment of men who do nothing but take care of the screens, and cost records are very carefully kept. The table brings out clearly the relative decrease of cost with an increase in the size of the plant in cities where the 8-hour day is in force.

TABLE XVIII

COST OF LABOR FOR OPERATING SCREENS AT CERTAIN STATIONS IN THE METROPOLITAN SEWERAGE DISTRICT

1908.

Station.	Average daily material screened. Mil. gals.	Size of screen opening.	Cost per million gallons.
Deer Island	59.8	1 inch	$0.14
East Boston	57.8	1 inch	0.14
Charlestown	31.3	1 inch	0.24
Ward Street	22.3	$\frac{1}{8}$–1 inch	0.53

At the Deer Island, Charlestown and East Boston stations, respectively, three screenmen are employed.

Strainers or Roughing Filters. The screening or straining of sewage through coke, or similar materials, for the removal of the coarse suspended matter, has been studied experimentally to a considerable extent, and has also been tried in a few places on a practical scale. The suspended matter which is removed from the sewage rapidly fouls the material of which the strainer is made, i.e., coke, coal, slag, cinders, peat, etc., so that this material, especially the top portion of it, has to be frequently removed and disposed of, either by drying and burning for fuel under boilers, or by some of the methods used for the disposal of screenings. In construction, a strainer is similar to a filter. The action which takes place in the former, however, is, in general, simply a mechanical one and independent of biological or chemical agencies.

At the Lawrence experiment station of the Massachusetts State Board of Health experimental studies of coke strainers using "Station sewage" were begun in 1894. At first the material used consisted of a layer of coke breeze about 6 inches in depth, containing more or less fine material. It was found necessary to remove about 10 cubic yards of this material for every million gallons of sewage strained. The depth of the strainer was increased to 12 inches and it was operated for some three years, with the result that 8 cubic yards of material had to be removed for each one million gallons of sewage strained.

On account of the fine dust in the material it was often found that clogging occurred in the underdrains. Later therefore there was used a strainer 15 inches in depth, containing coke from which dust and fine material had been removed. This was operated for over two years, during which time it was necessary to remove only $\frac{4}{10}$ of a cubic yard per million gallons of sewage strained. The strainers above described were operated at a rate of about 1,000,000 gallons per acre per day. The removal of organic matter was 34 to 50 per cent, on the basis of oxygen consumed.

The experiments at Columbus, Ohio (Johnson, 1905), included studies of coke strainers and the results obtained were very similar to those recorded at Lawrence. The general effect of the process is indicated in the table below. It was found impossible to burn the clogged coke without first subjecting it to a thorough drying process, and drying was accompanied by the production of extremely offensive odors. Furthermore, the efficiency of the strainers fluctuated considerably, as a result perhaps of breaks occurring in the straining surface.

TABLE XIX

EFFICIENCY OF COKE STRAINING AT COLUMBUS, OHIO

(Johnson, 1905)

Tons of dry solids per acre.

	A	B
In applied sewage	265	263
Removed by strainer	169	213
Subsequently removed from surface of beds	45	49
Subsequently found in the body of the strainer	117	151
Presumably dissolved through septic action	3	10
Passed out in the effluent	96	50

Perhaps the most instructive attempt to use strainers on a practical scale has been made at Gardner, Mass., where coke strainers were installed for clarifying sewage, and before treatment on intermittent sand filters. The quantity of sewage treated daily amounts to about 575,000 gallons and represents a tributary population of 9000. The coke strainers comprise four units of $\frac{1}{8}$ acre each, the total area being $\frac{1}{2}$ acre. They are constructed of an 8-inch layer of coke breeze (effective size, .41 mm.), supported by 6 inches of broken stone graded from $\frac{1}{2}$ to 1 inch. Underdrains are laid 6 feet apart in the stone. It was planned to distribute the sewage over the coke from perforated wooden troughs and an automatic apparatus was installed to dose the strainers on the contact bed principle. This apparatus was never very successfully operated, and was only used for a few years.

The sewage was afterwards run onto two of the four strainers for a week or so at a time. It stood about a foot deep over the coke, and while a portion was strained the larger portion overflowed through a pipe discharging into the underdrains, —provided for the purpose when the strainers first failed to work properly. The two strainers used were rested about once a fortnight, and the sewage was diverted to the other two. Drying took several days, and it was often necessary to use the beds again before they were cleaned. The strainers almost entirely failed to work in winter, and became easily clogged in the spring and summer months, so that they were of very little practical use.

CHAPTER IV

PRELIMINARY TREATMENT OF SEWAGE BY SEDIMENTATION

General Objects of Sedimentation. One of the most serious problems in sewage disposal is presented by the suspended solid material, partly mineral and partly organic in nature. The nitrogenous organic matter tends to form foul deposits in disposal by dilution, and both organic and inorganic suspended solids seriously increase the cost of all systems of disposal by filtration. To some extent in almost all plants, and to a great extent in highly developed and specialized plants, the physical factor of sedimentation is called in to remove the heavier portions of this suspended matter before the true processes of chemical and biological purification are brought into play.

Sedimentation is conditioned by the velocity of the liquid in which it takes place, by the size and specific gravity of the particles involved and by the time during which the force of gravity is allowed to act. In discussing the theoretical process of sedimentation with respect to the clarification of turbid river waters Hazen (1904) has constructed a diagram showing the relation between a given period of sedimentation (in terms of time required for one particle to settle from top to bottom) and the corresponding percentage of particles remaining in suspension. This mathematical discussion was of necessity based on very broad assumptions, one of which was that all particles of sediment in the water have the same hydraulic subsiding value; i.e., settle at the same rate.

The sedimentation of the suspended matter in sewage is susceptible of much less accurate mathematical analysis than is possible with suspended matter in water, for the reason that the former is constantly changing in shape and hydraulic subsiding value, as a result of the marked bacteriological and

chemical changes which are continually taking place. Furthermore, much of the suspended material in sewage consists of colloidal matter or suspended matter in a very finely divided state.

The general method relied upon in sewage sedimentation is of course a reduction of the velocity of flow by introducing a chamber of larger dimensions than the sewer itself; and the most important fundamental variables are the capacity of this chamber, governing the storage period, and its length, governing the velocity of flow. The critical velocity may be broadly defined as that velocity which, with a given period of flow, will attain the degree of purification desired in the particular plant in question. Fuller (1909) points out that when sewage is disposed of by dilution the velocity in the sedimentation basins should be at least as low as that in the currents into which the effluent is to be discharged. In preparation for filtration the process should be carried only far enough to remove as much of the suspended matter as can be disposed of more cheaply in this manner than by discharging it on the filters.

Types of Sedimentation. Sedimentation in the broadest sense may first be divided into Plain Sedimentation and Sedimentation aided by Chemicals. Plain Sedimentation tanks again may be designed to deal only with the grosser and heavier particles or they may be planned so as to remove a large proportion of the finer suspended organic matter as well. According to this difference in aim we may distinguish between Grit Chambers and true Sedimentation Tanks.

In the construction of sedimentation tanks there is again a radical distinction in plan between two types, the shallow tanks, with mainly lateral flow, and the deep tanks (of what is frequently called the Dortmund pattern), with mainly an upward flow.

Finally there are important modifications in the operation of sedimentation tanks which practically constitute independent processes. If the sludge, instead of being frequently removed, is allowed to remain in the bottom of the sedimentation tank so

that a considerable portion of it is liquefied by bacterial action, the tank is known as a Septic Tank. In connection with sedimentation tanks there should also be mentioned the Slate Beds of Dibdin. These slate beds, though usually classed as filters, are in fact made up of a series of small sedimentation tanks, (each having a capacity of only a few cubic inches) in which the solid matter from the sewage is deposited and allowed to undergo aerobic bacterial changes.

The present chapter deals only with the various forms of Plain Sedimentation. Chemical Precipitation will be discussed in Chapter V, and Septic Tanks in Chapter VI. The Dibdin Slate Bed, although practically a method of preliminary treatment for the removal of suspended solids, will be taken up later in Chapter X for the reason that it was historically an outgrowth of the contact process and because the theory of the contact bed is necessary for a proper comprehension of its mode of action.

Grit Chambers or Detritus Tanks. Grit chambers are used for the purpose of removing from the sewage, by sedimentation, the coarser and heavier suspended material of mineral character, or "road detritus." Such material consists largely of surface washings from streets, and hence grit chambers are most necessary where the sewerage system is on the combined plan. In fact their use with a system of strictly domestic sewers may cause an objectionable accumulation of foul organic matter which could more economically and satisfactorily be taken care of elsewhere. Grit chambers are often installed in connection with screening apparatus. They should be of relatively small capacity, and the velocity of flow should be so great as to permit but little deposit of organic matter.

On the other hand, it is important, especially if the sedimentation tanks are to be operated on the septic plan, to have the grit chamber large enough to remove the sand and road detritus which would otherwise rapidly fill the sedimentation tanks and interfere with the decomposition of the sludge by bacterial action. Where, as in the case of precipitation tanks, the sludge

is to be regularly removed and pressed, the thorough preliminary removal of detritus is not so important, as the presence of a little grit in the sludge facilitates pressing. On the other hand, too much sand and gravel, mixed with the sludge, may make it impossible for the main body of sludge to flow by gravity; and furthermore, this material may injure the sludge pumps, should such be used.

In order to meet these requirements, the dimensions of a grit chamber are generally such as to allow for a storage period of two or three minutes, with a velocity reduced to not less than five feet per minute. The chamber should be designed so that the detritus may settle as evenly as possible along the bottom, and not form a heavy deposit at the inlet end and leave but little at the outlet. Theoretically, therefore, the cross section should increase toward the outlet end. All grit chambers should be in duplicate, in order that one may be put out of service for cleaning without interference.

A good type of grit chamber, shown in Fig. 20, was installed in 1904 at the Worcester Massachusetts sewage plant. It consists of two parallel channels, each of which is 40 × 10 feet, and 9 feet deep. One half of the structure can therefore be placed out of service for cleaning.

The accumulated detritus at Worcester is removed by shoveling, and this is the common practice in the United States. In many of the German and English works the material is dredged out by mechanical means, (see Fig. 11), and at several places in Germany perforated metal vessels are fitted to the interior of the tanks and hoisted by cranes when full. The amount of detritus varies very widely with the amount of surface water admitted to the sewer system and is markedly influenced by storms. At the Dorchester pumping station of the Boston Main Drainage works, in 1907, there was removed from 32 billion gallons of sewage 578 tons of screenings and 5306 cubic yards of fine detritus. The latter, the detritus alone, works out at .17 cubic yards per million gallons. Assuming for the screenings a weight of 60 pounds per cubic foot, the total amount of

screenings and detritus together would be .19 cubic yards per million gallons.

At the Technology Experiment Station, on this same sewer system, a detritus tank was arranged to give a velocity of 12 mm. per second (2.4 feet per minute) and careful records of

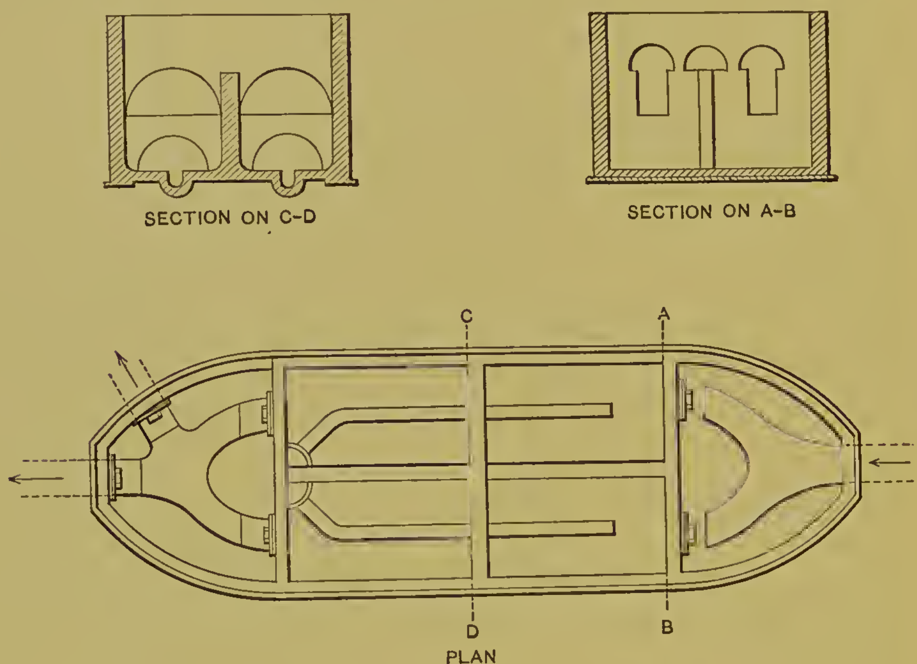


FIG. 20. Grit Chamber at Worcester, Mass.

the amount of sediment were kept. The total detritus removed by the grit chamber amounted in sixteen months to 4800 pounds, or 2.4 cubic yards, of wet material from a total volume of sewage equal to a little over 3 million gallons — 1600 pounds, or 0.65 cubic yards per million gallons. All the material thus removed was carefully sampled and its moisture determined. A portion of each dried sample was preserved and mixed with proportionate parts of later samples, and the mixture was finally analyzed. The amount of moisture, the proportions of clean stone and of dry detritus, and the analysis of the latter, are shown below, first in total amounts and then in parts per million of the total volume of sewage (3 million gallons).

TABLE XX

AMOUNT AND COMPOSITION OF DETRITUS REMOVED FROM GRIT CHAMBER AT BOSTON, MARCH 26, 1904, TO JUNE 1, 1905

(Winslow and Phelps, 1906)

	Wet Detritus.	Water.	Clean Stone, etc.	Fine, Dry Detritus.			
				Total.	Loss on Ignition.	Organic Nitrogen.	Oxy. Cons.
Total pounds	4,800	1,300	570	2,900	319	6.6	5.1
Pounds per million gallons of sewage	1,600	430	190	970	106	2.2	1.7
Parts per million parts of sewage	190	52	23	117	13	.26	.2

The figures show only the total and average amounts of detritus collected for the whole period. No accurate data are at hand to show the effect of the seasons on the amount of detritus deposited. The Boston city records, already referred to, show that at Moon Island there is a maximum of deposit in March, and a second maximum during the summer months. Local conditions of rainfall evidently largely determine the amount of detritus sent into the sewer, but monthly variations from the mean yearly deposit are not as a rule greater than 25 per cent of that value. At the Technology Experiment Station, in the spring of 1904, a large amount of snow was thawed by the warm rains, and during ten days 1600 pounds of detritus were taken from the detritus chamber and from a storage tank into which some excess of detritus had been carried over. This gave an average of 16,000 pounds of detritus per million gallons of sewage, and for shorter periods the rate of deposit might be greater still.

At Manchester, England, where the combined sewers are not provided with catch basins, the detritus may amount to 300 tons in a single day after a heavy rain.

The Sedimentation of Finer Suspended Solids. The line of demarcation between the so-called detritus and the remaining suspended solids, or sewage sludge proper, is rather sharply marked by the rapidity with which the particles settle. One

class of material will settle out in a very few minutes when the velocity is still considerable, and the other will settle only when the liquid is practically at rest and in the course of hours rather than minutes. According to figures given by Robinson (1896), a velocity of 0.5 feet per second will not move fine clay and 0.7 feet will just move coarse sand. Hence it may be stated that detritus may be removed from sewage at any velocity less than the former figure. On the other hand, sedimentation of the true suspended solids necessitates a slackening of velocities to 0.1 feet per second or less. In the London settling basins velocities of 0.07 feet are maintained; at Manchester, England, 0.05; at Saltley and Sutton, England, 0.03; and at Frankfurt, Germany, 0.01 to 0.02.

Important investigations in regard to the sedimentation accomplished with various rates of flow have been carried out by Bock and Schwarz at Hanover, Steurnagel at Cologne, Watson at Birmingham and Johnson at Columbus, Ohio.

Bock and Schwarz experimented in 1899 with tanks 162 and 243 feet long at velocities ranging from .014 to .067 feet per second. At a velocity between .014 and .027 feet per second they succeeded in depositing 55.7 per cent of the suspended solids in the shorter and 61.5 per cent in the longer tank. On increasing the velocity to .067 feet the efficiency of the longer tank was only decreased to 57 per cent. Complete quiescence for 24 hours produced a reduction of 88.8 per cent.

Steurnagel (1904) found that velocities less than .07 feet per second permitted results as good as any economically attainable. Complete quiescence for 12 hours only removed 84 per cent of the suspended solids: at a velocity of .014 feet per second 72.3 per cent was removed, and at a velocity of .070 feet, 69.1 per cent. On the other hand a further increase to .140 feet diminished the reduction to 58.9 per cent.

It is clear that while a good proportion of the solid matter in sewage will settle out quite rapidly, a certain fraction is too finely divided to be removed in this way under any practical conditions. The same thing was indicated by Steurnagel's

studies of quiescent sedimentation. In some experiments on this point in a 40-cm. cylinder the removal of suspended solids by sedimentation was about 25 per cent in five minutes, 50 per cent in thirty minutes, and 75 per cent in twenty-four hours. Steurnagel studied the same phenomenon in a deeper layer (2 meters) and found the removal of suspended organic matter to be 42 per cent in five minutes, 61 per cent in twenty-five minutes, 75 per cent in six hours and 80 per cent in twenty-four hours.

With continuous flow tanks the storage period must of course be somewhat longer than with quiescent sedimentation; and longer tanks are necessary with high than with low velocities. The suspended matter is acted upon by two forces, the force of gravity and the force due to the velocity of the moving sewage. The resultant of these two forces, shown graphically, should intercept the inside of the tank at a point below the outlet. Nevertheless it is probable that practice has erred in the past on the side of using too slow velocities and too long storage periods. In many cases money might have been saved with no loss of efficiency by using shorter tanks of less gross capacity.

As indicated by the table below, a more rapid rate of flow has also the advantage that the sludge deposited at the higher velocities contains the heavier solid matters, with less moisture, or, in other words, is more condensed than the sludge deposited at low velocities, and hence may be more easily handled.

TABLE XXI

COMPOSITION OF SLUDGE DEPOSITED AT DIFFERENT VELOCITIES

(Dunbar, 1908.)

Velocity in feet per sec.	Sludge (gallons).	Analysis of sludge.	
		Moisture (per cent.).	Dry residue (per cent.).
.014	4.040	95.57	4.43
.070	2.474	92.87	7.13
.140	1.838	91.34	8.66

Construction of Sedimentation Tanks. Sedimentation Tanks for sewage are operated by two different methods,— continuous and intermittent. With intermittent subsidence, where tanks are filled with sewage and then allowed to remain full in a quiescent state, there is danger of stirring up the sludge at each emptying and filling of the tank; it is necessary to use a floating or movable outlet, which is frequently unsatisfactory; and there is sometimes absorbed by this method an available head which could be used to better advantage. Continuous subsidence is therefore usually employed.

The principal features to be considered in the design of a sedimentation tank are total capacity, proportions in relation to velocity of flow, and number of units. Minor details of importance are the slope of bottom, the design of floor and walls, inlets and outlets and baffles.

It is impossible to state in definite terms the proper capacities for sedimentation tanks. Roughly speaking, the most approved practice fixes the capacity of such tanks at from 4 to 12 hours flow, depending somewhat upon the character of the sewage, since the kind of suspended matter plays an important part in determining the proper capacity. If the suspended matter is largely of a mineral character, it will of course settle much more readily than if it is flocculent and about the specific gravity of water.

Except when used for special purposes, shallow sedimentation tanks are usually built rectangular in both plan and section, as this form affords the greatest economy of construction consistent with efficiency. By first deciding upon the total capacity and velocity of flow, the shape of the tank is largely determined; that is, there must be a definite cross-sectional area in order to correspond with the given daily flow and a given velocity. It remains, however, to decide upon the proper depth, which in turn determines the width. The depth is sometimes controlled by local topography and facilities for disposing of the sludge; but depths of 8 to 12 feet are, in America, considered good practice, the deeper tanks being used when

they are to be operated on the septic plan. In England a depth of 4 to 8 feet has been recommended by engineers of authority, in the belief that a more even distribution of flow will be obtained in a tank of 8 feet or less. As relatively rapid velocities are desirable, the length of the tanks should be at least four times the width. Tanks longer than this would involve unduly expensive construction, but high velocities and long lengths of travel can be obtained by constructing several parallel units to be used in tandem.

In deciding upon the number of tank units to be provided, fluctuations in flow should first be considered. If the dry-weather flow is very greatly increased at times by surface water, then a tank which affords the proper sedimentation period and velocity of flow in dry weather will be entirely too small in times of storm, with the result that the " critical velocity " will be exceeded and suspended matter will be carried out with the effluent. By installing a number of units, some may be held in reserve for the storm flows; or, if all of the units are used in tandem during dry weather, the storm flows could be handled by operating all in parallel.

The advantage of dividing tanks into units is particularly apparent where purification works are planned to allow for a future increase in population. In such cases the sewage flow will be very small at first and will increase gradually during the first few years that the system is used. Another advantage of separate units lies in the facility for cleaning and the avoidance of large quantities of sludge which must be handled at one time. All works, no matter how small, must have at least two units, and plants of considerable size should have many more.

Both experiment and theory would indicate that a tank for plain sedimentation should be deeper at the inlet end, in order to provide space for the deposited material without unduly decreasing the cross section of the tank. The stirring up of the solids deposited in the tank by temporary increase in velocity of flow is also avoided by such design.

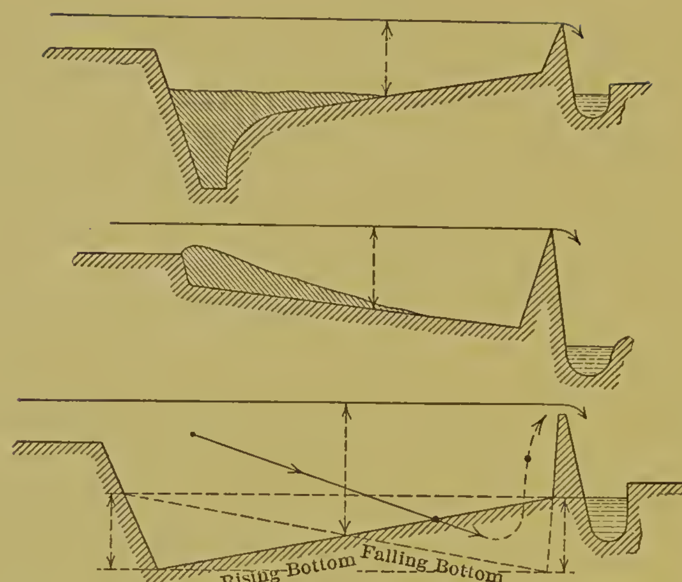


FIG. 21. Diagram of Efficient and Faulty Tank Construction (after Steurnagel, 1904).

The floors of sedimentation tanks are almost universally built of concrete, as this material may be readily adapted to the desired shape of the floor. The cost of concrete is less than that of any other satisfactory material. Usually a thickness of

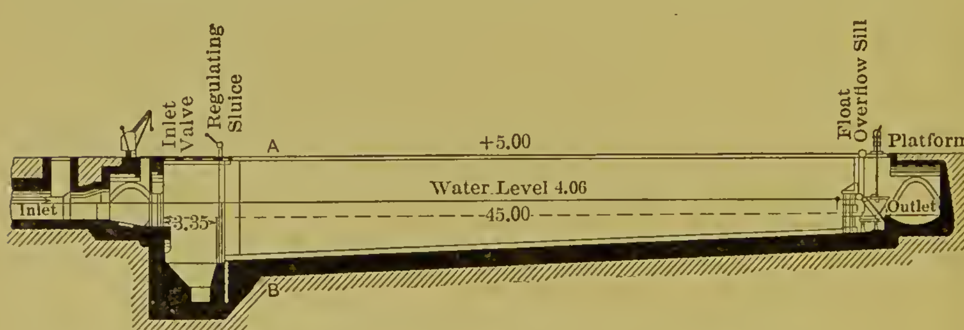


FIG. 22. Sedimentation Tank at Cologne (after Steurnagel, 1904).

6 inches will afford a sufficiently water-tight floor, although in some cases, where the ground is soft, it may be desirable to increase the thickness or to use steel reinforcement. The durability of concrete where exposed to sewage has never been questioned until very recently, when there have been reported

instances of the disintegration of concrete tanks, presumably from the action of the sewage gases. (Eng. News, 1910.)

In order to facilitate the removal of sludge, the floor should have a slope of at least 1 in 60 toward an outlet provided for draining the tank, or toward outlets for draining the subdivisions. Even with the slope above mentioned, it will be found impossible to drain the tank completely without pushing out some of the sludge by hand or flushing it out with a hose stream. Such a procedure, however, is entirely practicable; and as slopes steeper than 1 in 50 tend to make the construction of the tank more difficult they are not recommended.

The design of the walls of sedimentation tanks is governed largely by the character of the foundation, the height above the original ground level and the length of the wall between supports. In all cases the walls should be water-tight, and when built of concrete special precaution should be taken to prevent leakage at the joints. It may sometimes be desirable to coat the inside of the walls with pitch or tar. It has been suggested that this will not only add to the water-tightness, but will tend to protect the concrete in case the sewage should have any undesirable effect upon it.

In general the walls should be proportioned according to the same principles that govern the design of dams and retaining walls. A tank wall, when the tank is empty, will constitute a retaining wall if placed below the original ground level or if banked with earth on the outside. The materials of construction are usually plain concrete, reinforced concrete or brick masonry. With all open tanks the tops of the walls should be wide enough for a man to walk upon safely, and to attain this end a coping may be used if necessary.

The simplest and most effective form of inlet and outlet for all kinds of sedimentation tanks is a weir with perfectly level crest, extending across the tank. With this form, and especially if the tank is to be operated on the septic plan, there should be scum-boards immediately in front of both inlet and outlet to prevent surface currents. The channel conveying

the sewage to the inlet weir should not be unduly large, or objectionable deposits may result before the sewage reaches the tank.

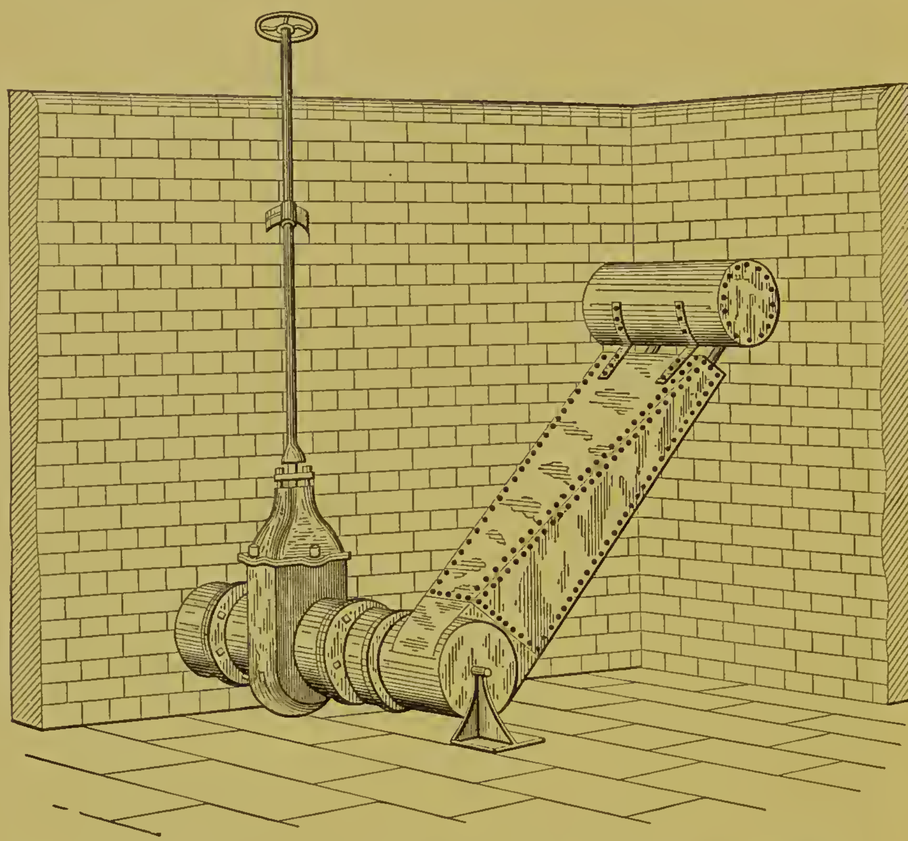


FIG. 23. Floating Arm for Emptying Sedimentation Tank (copied by permission from Santo-Crimp, 1894).

With large plants this weir construction is open to objection from a constructional standpoint and, in such cases, an even distribution of the current is obtained by discharging the sewage into the tank, and drawing it off, through several openings, generally below the surface, and by placing a baffle or deflector within a few feet of the end walls.

Submerged walls and suspended baffles, or scum-boards, for preventing the forward movement of suspended matter form an important part of sedimentation tank construction. The submerged walls serve to subdivide the tank in such a manner as to

facilitate the removal of sludge. They also guard against temperature effects by preventing direct currents between inlet and outlet. Schmidt has demonstrated, by means of coloring matter, that during the cooler portions of the year the warm sewage tends to remain on the top of the cooler contents of the tank.

A point of great importance in the construction of sedimentation tanks is the provision of some means for the removal of the sludge. This must be accomplished at rather frequent intervals, particularly in summer, if putrefactive decomposition is to be avoided. In hot weather it may be necessary to remove sludge every six or seven days, while in cold weather it may be kept for a month or more. Some arrangement must be made for drawing down the supernatant liquor to the level of the sludge deposit so as to keep at a minimum the volume of sludge to be handled. This may be accomplished by means of arms on the outlet pipe so adjusted that the sewage will at all times be drawn from a level a few inches below the surface (see Fig. 22). Another device for accomplishing the same purpose consists of a vertical standpipe provided every 1 or 2 feet with openings, controlled from above, through which the tank contents may be drawn off at given levels.

When the tank effluent is to be discharged intermittently, onto sand or other filters, it may be desirable (in order to avoid building a dosing tank) to install some automatic controlling device which will hold back the flow until the sewage level has risen to a certain height and then allow it to discharge at a rate greater than the rate of inflow. With such a device the sewage, after rising in the tank to the desired level, feeds into a float chamber through a siphon. This causes the float to rise and open a butterfly valve controlling the main outlet of the tank. When the sewage level drops the float lowers and closes the outlet.

In any case the sludge is finally allowed to run off from a special pipe at the lowest point and the heavier sludge must generally be worked down to the outlet by hand. At some of

the English plants interesting mechanical appliances are installed for moving sludge to the tank outlet. A device of this sort in use at Bolton is described in Chapter VII.

Sedimentation in Deep Tanks. An entirely different type of sedimentation basin is the deep tank with a conical bottom, in which the sewage flows upward and the sludge is removed from the bottom while the tank is in continuous operation. The first important tank of this kind, intended for use without the application of chemicals, was built at Dortmund, Germany, some twenty years ago. For this reason the name " Dortmund " has been applied to tanks of this design.

The principal advantage claimed for this type of tank lies in the fact that sludge may be removed without stopping its operation. On the other hand, the sludge drawn off from such tanks is very liquid. Deep tanks are generally more expensive to construct than shallow ones; and in their practical operation it has been found difficult to prevent the sludge from adhering to the sides of the conical bottom as well as other portions of the tank. Frequent cleaning is necessary if the tank is to be successful, and in England various kinds of revolving scrapers have been designed for this purpose.

Within the last few years the deep tank has come into great favor as a method of treating the effluent from trickling beds. The suspended matter in such an effluent is flocculent and easily removed, and since it is at the same time of a less putrescible character the danger of undue putrefaction is diminished.

At Birmingham two sets of deep tanks are used for treating the septic effluent before filtration and the trickling effluent after filtration. The first tanks (receiving septic effluent) are cylindrical above and conical below, as shown in Figs. 24 and 25. The sludge collects at the bottom of the cone and enters there a pipe, inside which it is forced up and out when the sludge valve is opened by the weight of the liquid in the tank.

The sewage is admitted by a vertical pipe at the center with its open end near the bottom, and the settled sewage overflows into an open trough across the center of the tank.

The so-called Separator Tanks, which treat the trickling effluent at Birmingham, are of slightly different construction, although the general principle is the same. The tanks are rectangular in plan, the side walls being vertical for a distance of about 7 feet from the coping, the lower portion being constructed in the shape of an inverted pyramid, and terminating in a small sump about 4 feet square and 2 feet 6 inches deep, from which the sludge is extracted. The inlet pipe enters at

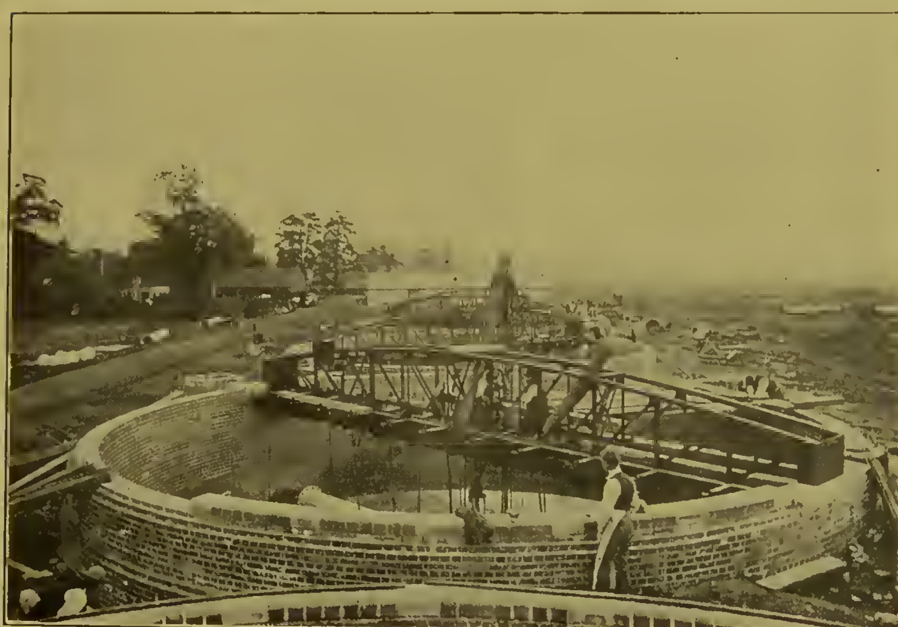


Fig. 24. General View of Deep Sedimentation Tank at Birmingham, England (courtesy of J. D. Watson).

the bottom of the vertical walls and is carried to the center of the tank, where it dips downward for a depth of 5 feet. The outlet is in the form of a weir, occupying the whole length of one side. The sludge is removed by means of a sludge pipe extending down the side of the tank and into the sump at the bottom. The outlet of this pipe is about 4 feet below the water level in the tank.

The water enters through the inlet pipe with a downward velocity of 1 to 2 feet per second, and as it spreads laterally, rises with a gradually decreasing velocity. Theoretically the upper

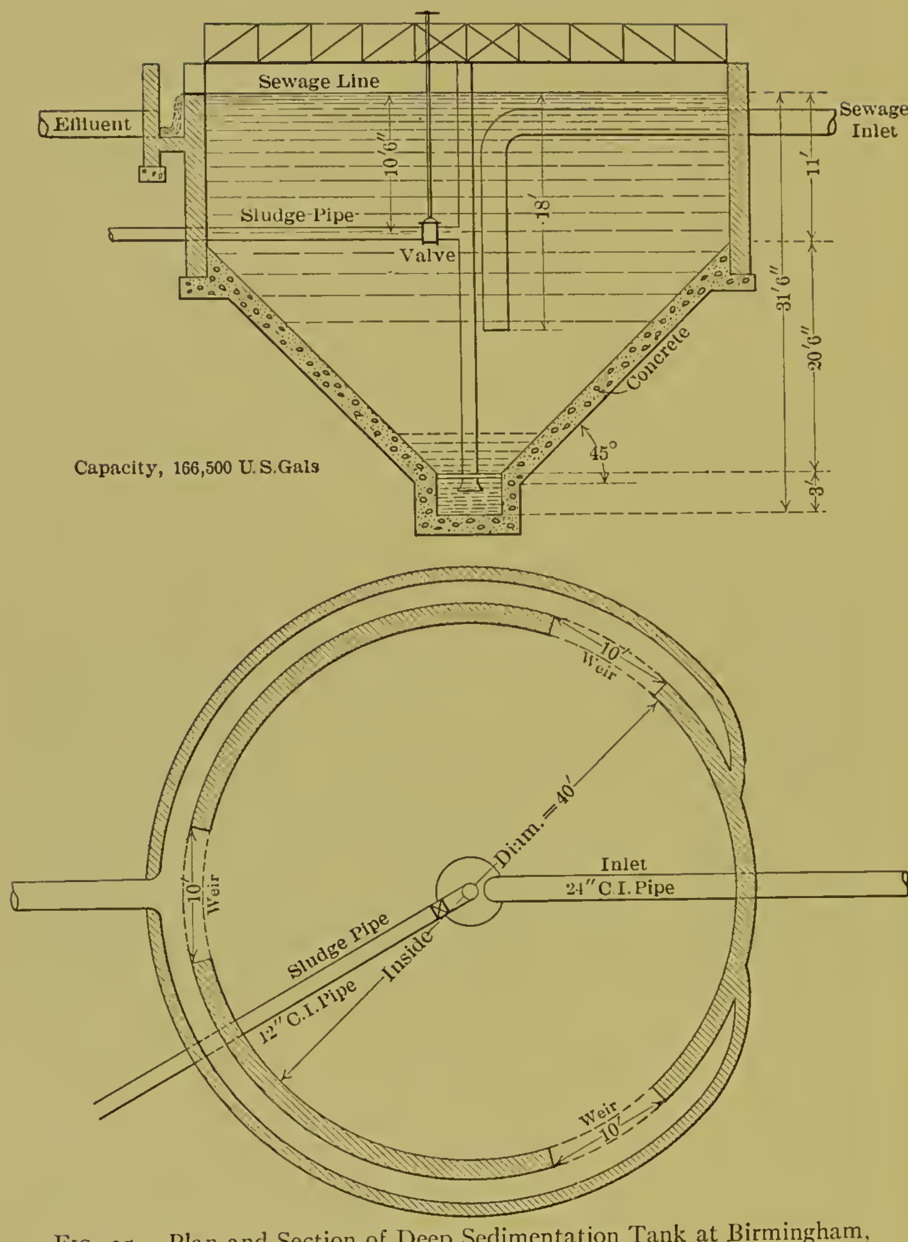


FIG. 25. Plan and Section of Deep Sedimentation Tank at Birmingham, England (courtesy of J. D. Watson).

part of the tank with vertical walls is not required, since the efficiency of such a tank depends solely on the surface area. It has, however, been found in practise advisable to make this portion 5 to 7 feet deep. About 85 per cent of the total suspended solids may be removed by a tank of this form.

Practical Results of Sedimentation. The efficiency of sedimentation will naturally vary widely, according to the nature of the particular sewage dealt with. At Birmingham, as just stated, the deep tanks remove 85 per cent of the suspended solids in trickling effluent, and Dunbar (1908) states that 80 to 90 per cent of the suspended matter in sewage can be removed. On the other hand, most English and American data do not indicate that any such results can be obtained with crude sewage. According to Fuller (1909): "From 50 to 65 per cent of the suspended matters in well-screened sewage can be removed by sedimentation in basins holding 6 to 12 hours, average flow. . . . A substantial proportion, say from 30 to 40 per cent of the suspended matter of all sewages, is so fine that it is not responsive to the laws of subsidence and in an engineering sense can be spoken of as colloidal matter."

The efficiency of a number of continuous-flow shallow sedimentation tanks in England and America is indicated in the table below, the original data being given in the Final Report of the Royal Sewage Commission and in Mr. G. W. Fuller's Cornell address:

TABLE XXII

EFFICIENCY OF SEDIMENTATION TANKS

Place.	Sedimentation period. Hours.	Rate of flow. Inches per minute.	Suspended solids. Parts per million.		Per cent reduction.
			Influent.	Effluent.	
Halton, Eng.	15.5	.66	177	107	40
Clifton, Eng.	5.3	1.70	490	240	51
Oswestry, Eng.	4.1	3.40	320	158	51
Dorking, Eng.	11.9	.31	208	101	51
Plainfield, N. J.	10.0	....	118	54	54
Columbus, O.	13.0	....	304	101	67
Reading, Pa.	15.0	....	165	42	75

The sludge accumulating in sedimentation basins usually amounts to 4 to 6 cubic yards per million gallons with American sewage; and the disposal of this sludge forms a separate and serious problem, which will be discussed in Chapter VII.

CHAPTER V

PRELIMINARY TREATMENT OF SEWAGE BY CHEMICAL PRECIPITATION

History of Chemical Precipitation. The failure of sedimentation to clarify sewage thoroughly, and the hope of financial gain, led to the introduction in England, during the early sixties, of what is known as the chemical treatment of sewage. It was claimed that by this method the putrefactive substances in sewage would be removed, and that, the nitrogenous compounds being precipitated, the sale of this precipitated matter as a fertilizer would pay for the cost of treatment and realize a substantial profit. The process was at first greeted on all sides as a method which would solve the sewage problem, and between 1880 and 1890 nearly all the sewage plants installed in England were chemical precipitation plants.

The hope of financial gain, however, was not realized, for though the precipitate contained nitrogenous compounds and a certain amount of phosphate, it also contained so much water — averaging about 94 per cent — that it was found impossible to prepare a marketable product at a cost at all comparable with the price of other chemical fertilizers. Further, it was also slowly recognized that though the process yielded a clear liquid, the liquid contained a large amount of soluble organic matter and was putrescible.

Notwithstanding these facts, however, the process continued in general use into the nineties, when the bacterial methods of purification, by which nonputrescible effluents could be obtained, began to receive serious attention from sanitary engineers.

The chemical treatment of sewage consists in adding to the sewage certain chemical compounds which cause a voluminous

flocculent precipitate to be formed, and allowing the sewage thus treated to remain quiescent in a large tank for a number of hours, or to flow continuously through a series of tanks at so slow a velocity that the insoluble compounds formed, together with the suspended matters of the sewage, settle to the bottom of the tank, leaving a clear supernatant liquid.

The chemical compound first used was lime, added to the sewage in the form of milk of lime. The action of the lime depends upon its uniting with the free and combined carbonic acid in the sewage to form insoluble calcium carbonate, which in settling out of the liquid carries down with it the suspended matter. The amount of lime used at different places has varied, depending on the character of the sewage, an acid sewage requiring much more lime than a neutral or slightly alkaline sewage to produce the desired effect. The best results were obtained by the addition of just sufficient lime to decompose the carbonates present in the sewage, i.e. from five to ten grains of lime per gallon. If too little lime was used, sedimentation took place very slowly, and a clear effluent could not be obtained; if too much lime was added, varying amounts of the suspended organic matter in the sewage were rendered soluble, causing the effluent to be more putrescible and more obnoxious than the effluent from plain sedimentation.

Besides lime, various other chemical compounds were tried, resulting in innumerable patents being taken out for the chemical treatment of sewage. Of these chemical compounds, lime and ferrous sulphate, or lime and aluminium sulphate, have been more extensively used than any others, on account of simplicity of application and superior effectiveness under ordinary conditions.

Experiments as to the effect of lime, iron salts and aluminium sulphate on sewage have been made by Hazen in America and Dibdin in England. Hazen (1890), working with the sewage of Lawrence, Mass., which contained but little trade waste, was alkaline to litmus paper, and gave on analysis in parts per million 18.3 parts free ammonia, 2.7 parts albuminoid ammonia

in suspension and 3.9 parts in solution, found that lime, ferrous sulphate and aluminium sulphate, when used in the correct amounts, removed practically all the suspended matter, and, judging from the albuminoid ammonia, from 20 to 40 per cent of the soluble organic matter. He found that a definite amount of lime gave a result as good as, or better than, either more or less lime; that the more copperas, ferric sulphate or aluminium sulphate used the better the result; that no lime was necessary with aluminium sulphate or ferric sulphate if the sewage was alkaline; that with lime alone the best results were obtained when the amount of lime added was just sufficient to decompose the carbonates present; that with either ferric or aluminium sulphate 3.5 grains per gallon, or 500 pounds to the million gallons, gave nearly as good results as larger amounts; that with ferrous sulphate and lime there was no great advantage in using more than 7 grains per gallon, or 1000 pounds of the iron salt per million gallons; that much better results were obtained by first adding lime and then copperas than by the reverse process; that ferric sulphate was preferable to ferrous sulphate, not only because it could be used without lime, but because the ferric hydrate was more insoluble than the ferrous hydrate and brought about quicker sedimentation.

Dibdin (1889), working on London sewage, obtained the best results with lime and copperas, 10 grains of each to an English gallon, all the suspended matter and 30 per cent of the soluble organic matter being removed. Lime used alone, 15 grains to the gallon, removed 25 per cent, and lime and alum, 5 grains of each per gallon, removed 18 per cent of the soluble organic matter.

The above experiments show that lime with either ferrous sulphate or aluminium sulphate, when used in proper amounts, will remove all the suspended matter and a certain amount of the soluble organic matter; and though the amount of soluble organic matter that is removed when working on a large scale is less than indicated by the above experiments, the general results are in accord with those obtained by Hazen and Dibdin.

It should, however, be stated that though iron salts and lime effect a more complete removal of organic matter than aluminium sulphate and lime, the effluent obtained from the use of iron salts is in certain respects less satisfactory than that obtained from aluminium salts. It always contains a certain amount of iron in solution, as iron forms soluble salts with organic acids which are not decomposed by the action of lime. The presence of iron salts in an effluent causes more or less trouble, especially when the effluent is further treated by bacterial methods. The soluble organic iron compounds undergo decomposition with the final formation of insoluble ferric oxide, which not only clogs the sand of a bacterial filter bed, but may close up the open joints of the underdrains.

Among the other chemical processes used may be mentioned the Spencer Alumino-Ferric, the International, the A. B. C., and the Webster process.

The Spencer Alumino-Ferric Process. This process consists in adding alumino-ferric, a mixture of ferrous and ferric salts with aluminium sulphate, alone or with lime, to the sewage.

The alumino-ferric is supplied in blocks of about 30 inches by 20 inches by $3\frac{1}{2}$ inches thick, and at small works these cakes are placed in cages and suspended in the sewage carriers.

This method was recommended by Dr. Tidy for Chiswick, England, 7 grains of lime and 5 grains of alumino-ferric per gallon, being said to give satisfactory results, and it has also been used in various other English towns.

The International Process. This process consists in adding a compound called "ferrozone" to the sewage and filtering the effluent through "polarite." Ferrozone is obtained by a patented process from an iron deposit, found in South Wales, and contains ferrous, aluminium, calcium and magnesium sulphates and magnetic iron oxide. When added to sewage the soluble portion of the ferrozone causes the formation of a precipitate, while the insoluble portion, the magnetic oxide of iron, aids subsidence.

The effluent is filtered through "polarite" also obtained by a patented process from the same iron deposit. It is hard, porous, and absorptive. This procedure is used in various places in England, and has also been tried in the United States.

The A. B. C. Process. This process, at one time very prominent as one of the chemical precipitation methods, was supposed to consist in adding to the sewage, alum, blood, and clay, but, as generally used, the substances were alum, clay, and carbon, 50 grains of the mixture per gallon of sewage.

The Webster Process. In this process the active precipitant was ferric hydroxide, obtained by passing a weak electric current through iron plates suspended in the sewage channels, the plates being connected alternately with the positive and negative terminals of a dynamo. This process was carefully studied by Sir Henry Roscoe and Mr. Alfred E. Fletcher, and their report shows that the effluent obtained contained less organic matter and was less liable to undergo secondary putrefaction than the effluent from any other precipitation process. The cost, however, seemed to prohibit it from being used on a large scale.

General Practice in Chemical Precipitation. Taking into account all the known facts, it would seem that the chemicals best adapted for the treatment of sewage are ferric sulphate and lime. Aluminium sulphate is preferable to ferrous sulphate, as it does not tend to form soluble salts with the organic matter; yet, in general practice, ferrous sulphate is ordinarily used, for as a rule it can be obtained more easily and cheaply than either of the other two salts.

The amount of ferrous sulphate added to sewage free from trade waste seldom exceeds 5 grains per gallon, although experiments have shown that the best results are obtained when the amount reaches 7 grains per gallon. In sewage containing trade waste, the amount of iron present is often so great that any further addition is unnecessary, and all that is required is the addition of lime. The amount of lime used depends on

the character of the sewage. It must be sufficient in amount to cause the sewage to be just alkaline after the iron or aluminium sulphate has been added. As has been said, if too great an amount of lime is added, it acts on the organic matter in suspension causing the effluent quickly to become offensive. The test usually applied at sewage works to determine whether the right amount of lime has been introduced, is to add a few drops of an alcoholic solution of phenolphthalein to the sewage. If the right amount of lime has been added a very faint pink color is produced; if not enough lime has been added no color is formed; if too large an amount, a very decided red or pink color appears. The amount of lime and iron or aluminium salts added varies widely at different places on account of the differences in the character of the sewage.

At London 3.3 grains of lime and 0.82 grains copperas are used for one United States gallon; at Salford, 7.5 grains lime and 2.5 grains copperas; at York, 4.7 grains lime and 3.6 grains aluminoferric; at Bolton, 7.83 grains ferrozone; at Leeds, 2.8 grains lime; at Sheffield, 4.2 to 6.7 grains lime; at Providence, R. I., 5.08 grains lime; at Worcester, Mass., 6.57 grains lime. One grain per gallon equals 17.1 parts per million.

The lime should be added to the sewage in the form of milk of lime. A soft fine-grained lime containing from 90 to 95 per cent of calcium oxide, which slakes easily, should be selected. It should be thoroughly slaked, and if possible a day's supply should be weighed out and slaked twenty-four hours before its intended use. Sufficient water or sewage should be added to the slaked lime to form milk of lime, as otherwise part of the slaked lime settles out of the liquid and is rendered inactive. When either copperas or aluminium sulphate is used, it should be dissolved in weighed amounts in a known volume of water contained in vats, so that the amount added to the sewage can be easily regulated by adjusting the flow of the liquid. The copperas or the aluminium sulphate should be added to the sewage below the point where the milk of lime enters, and to insure a thorough mixture of the chemicals with the sewage,

the sewage should be run through a channel containing baffle boards before it enters the tank.

Fig. 26. Mixing Channel with Baffle Boards, Worcester Sewage Works.

The chemical reactions that take place are as follows: the milk of lime, calcium hydroxide, first reacts with the free carbonic acid and the acid carbonates contained in the sewage, insoluble calcium carbonate being formed, and if mineral acids are present it neutralizes them and renders the sewage alkaline. When the copperas or aluminium sulphate is added to this alkaline sewage the excess of calcium hydroxide decomposes the salt with the formation of ferrous or aluminium hydroxide and calcium sulphate. The calcium carbonate and the ferrous or aluminium hydroxide that are thus formed are heavy insoluble substances, which in settling out of the sewage carry down the suspended matter, and by occlusion, or by the formation of insoluble colloids, a small amount of the organic soluble substances.

Settling Tanks. Both the shallow rectangular tanks and the deep Dortmund tanks, described in Chapter IV, are used in the chemical treatment of sewage, and as chemical treatment is practically only sedimentation aided by the addition of chemicals, what has been said regarding the construction and use of settling tanks in the chapter on Sedimentation applies to tanks used for chemically treated sewage. The Dortmund tanks, however, have not as yet been employed for this purpose, either in England or America, but in Germany they are considered especially adapted for handling the sludge from chemical precipitation.

Rate of Flow. In the early days of chemical precipitation it was customary to allow the sewage after the addition of chemicals to remain quiescent in a tank for six hours, then to draw off the clear liquid and again fill the tank with sewage. There is, however, as has been stated in Chapter IV, a serious objection to this method, in that it is almost impossible not to stir up the sludge when the tank is emptied or filled, unless the sludge is removed each time the tank is emptied. Consequently, this procedure has been generally superseded by the method of continuous flow, and though very little experimental work has been done to determine the rate of flow best adapted to sedimentation when chemical substances have been added to the sewage, general practice indicates that with shallow tanks a period of six to eight hours gives the most satisfactory result, while with deep tanks, a period of only two to three hours is required.

Formation and Removal of the Sludge. The amount of sludge formed by chemical precipitation is at least fifty per cent more than the amount formed by plain sedimentation, although it depends more or less on the character of the sewage and the chemicals used.

At London about 953 cubic feet are produced per million gallons of sewage; at Salford, 1233; at Leeds, 820; at Sheffield, 383.5; at Providence, R. I., 525; at Worcester, Mass., 659 cubic feet.

The sludge is a slimy mass with a specific gravity of about 1.04 to 1.06, and must be removed from the tanks before active

putrefaction sets in, as otherwise the gas formed brings to the surface of the liquid a greater or less amount of the deposited substances, which are then carried away in the effluent. This in warm weather, often necessitates the removal of the sludge once in every seven to ten days. For the purpose of removing the sludge from shallow tanks the supernatant liquid is run off by use of a floating arm, or by means of a series of superim-

Fig. 27. Hand-propelled Squeegee used at Salford, England.

posed valves as described in Chapter IV. Though the sludge contains 90 to 95 per cent water, only a portion will flow of itself to the lower end. of the tank or through the sludge channel, which is often made in the cement concrete of the bottom of the tank, and the flow must be assisted by the use of hand or horse power propelled squeegees. (Fig. 27.)

The Ashton Mechanical Squeegee. At Bolton an ingenious mechanical device, known as the Ashton mechanical squeegee, is used. (Fig. 28.) It is constructed of angle iron, having grooves

along the front into which one-inch boards can be placed. The bottom boards and ends have facings of rubber. The machine is run on two light iron tracks set in the concrete at the bottom of the tank. To work the mechanical squeegee successfully the supernatant liquid is drawn off down to the sludge by means of a floating arm or other valve device. Sufficient

FIG. 28. Ashton Mechanical Squeegee (by permission of Mr. Ashton).

clarified sewage from the neighboring tank is then run in behind the machine to move both the sludge and machine to the bottom of the tank, the fall being 1 in 80. To move the squeegee back to the upper end of the tank, sewage is run into the clean tank, and as the volume increases sufficient power is obtained to push the machine backwards to its original position at the top end of the tank, to be again ready for removing the sludge.

With deep tanks it is not necessary to remove the liquid, the sludge being drawn off from the bottom of the tank, or raised through an iron pipe by the pressure of the overlying liquid, or

by pumping. There is more or less difficulty, however, in preventing the adhesion of some of the solid matter to the sides of the tank, leading to putrefaction which may give more or less of a septic character to the liquid. The sludge obtained by chemical treatment of sewage has the same characteristics and properties as the sludge from plain sedimentation or from the septic tank process, and consequently the question of the disposal of this sludge is considered in Chapter VII.

Results Obtained by the Chemical Treatment of Sewage. The chemical treatment of sewage removes practically only the suspended solid matter. Under favorable circumstances and with careful work a certain percentage of the soluble organic matter may be carried down, but there is no workable precipitation process which will give a nonputrefactive effluent, and chemical precipitation must be considered merely as a method of removing from the sewage visible solid matter. The amount of purification accomplished by chemical treatment is indicated in the following table:

TABLE XXIII.

EFFICIENCY OF CHEMICAL PRECIPITATION

Parts per Million

	Suspended Solids.		Albuminoid Ammonia.	
	Sewage.	Effluent.	Sewage.	Effluent.
Leeds	775	96	7.88 Total	5.09 Total
Bolton	496	176	9.7 "	3.4 "
Bradford	840	36	32.8 "	17.80 "
Sheffield	417.5	103.4	9.02 "	5.68 "
Salford	466	30	....	.... "
York	212	89	8.2 "	5.4 "
Providence	397	47	8.86 " 4.28 Dissolved	4.80 " 4.15 Dissolved
Worcester	231	68	7.33 Total 3.34 Dissolved	4.66 Total 3.65 Dissolved

Special Methods of Chemical Treatment. When the sewage contains very large amounts of certain trade wastes, special chemicals may be used with excellent effect. A typical case of this sort is the sewage of the city of Bradford, England. The

sewage contains considerable quantities of wool grease, and the treatment consists in passing the sewage through detritus tanks, screening, and adding sulphuric acid, 30 to 40 grains H_2SO_4, 1.82 sp. gr., per United States gallon. The sewage is passed through settling tanks used in series. The sludge, containing 80 per cent water and 8.57 per cent grease, after removal from the tanks is treated with a further quantity of sulphuric acid, heated to 100° C. and run through filter presses. The grease obtained is said to pay for the tank treatment of the sewage.

Chemical Precipitation Plants in Great Britain and the United States. Though chemical precipitation can only be considered, as a method of preliminary treatment, its use as a final process may in certain cases be permissible, as where the effluent is discharged into so large a volume of water that the removal of suspended matter constitutes a sufficient purification.

In Great Britain, London and Glasgow are the two largest cities which use chemical precipitation as a final process; and in the United States, Providence and, to a limited extent, Worcester follow the same method. In England it is also used in a number of places as a preliminary process, and at Leeds the septic tank method of clarification has been abandoned and chemical treatment is to be instituted in its place.

Chemical Precipitation at London. The sewage of the city of London was originally discharged from various sewers directly into the Thames, producing an unbearable nuisance, mentioned in Chapter II. To cope with these conditions, in 1858 the Metropolitan Board of Works, with Sir Joseph W. Bazalgette as engineer, began the construction of intercepting sewers on both sides of the river. The Crossness outfall sewer was completed in 1862, and those at Barking in 1863 and 1864. Covered reservoirs were built at both Crossness and Barking, one of 30,000,000, the other of 42,000,000 U.S. gallons capacity. These reservoirs were designed to store the sewage for discharge on the outgoing tide. The capacity of the reservoirs soon proved insufficient, and on account of numerous complaints as to the condition of the river, after a long inquiry it was decided to adopt

chemical precipitation as the method of treatment, using lime and copperas as the precipitants. The Barking precipitation works were put into operation in 1889 and the Crossness works in 1892, and, according to Baker, (1904):

" The sewage comes to each works well broken up by its flow of some 15 to 20 miles, besides which, a portion of the sewage coming to Barking has been pumped twice, and all the sewage at Crossness is pumped once, while a portion of that at Crossness has been pumped before it reaches the works. At Crossness all the sewage is screened before it is pumped. At Barking, at the present time, the sludge only is screened from the settling channels.

" There are 13 brick-covered precipitation tanks, or channels, at Barking, ranging from 860 to 1210 feet in length, and having a width of 30 feet, an average working depth of 8 feet, and a combined holding capacity of a little more than 20,000,000 Imp., or 25,000,000 U.S. gallons. The effluent from the channels passes into the old 42,000,000 gallons tidal reservoir, and thence through nine openings into the river. The tanks are operated separately, on the continuous flow plan. There are also twelve sludge settling tanks, each 20 by 140 feet by 13 feet effective depth. Since 1894 three elevated steel sludge tanks have been built at Barking. Each tank has a capacity of 1000 long tons, or one shipload. Normally, the sludge is pumped to the boats, but the tanks are kept full for the quick loading of boats that come in near nightfall and for loading on Monday morning.

" At Crossness the old tidal reservoir was converted into four covered precipitation tanks, each 560 feet long and 128 feet wide, and two new covered tanks were built, each 558 feet long and 99 feet wide. The combined holding capacity of these six tanks is about 21,700,000 Imp., or 26,000,000 U.S. gallons. There are also eight sludge settling tanks, 130 by 22 feet, with a working depth of 15 feet.

" Six sludge ships were put in service from 1887 to 1895, inclusive. They are approximately 230 feet long, 38 feet broad, 14 feet deep, and on trial developed from 950 to 1250 horsepower each. The two earlier boats are loaded through hatchways and the four later ones are loaded through a central hopper, from which inlets controlled by valves lead to the four

sludge compartments, formed by a longitudinal and a cross bulkhead. The load of 1000 long tons can be discharged through eight valves in six minutes, but as a rule each sludge load is distributed over a course taking an hour, at a speed of ten knots. As a rule, five boats are in service at a time, and one is being overhauled."

The total amount of sewage treated per day at the present time is about 280,000,000 U.S. gallons, 3.3 grains of lime and 0.82 grains of copperas per U.S. gallon being added. The daily production of sludge equals 8200 long tons, and the cost, as given by Baker (1904), of treating the sewage and conveying the sludge to sea is about $8.67 per million U.S. gallons.

As the result of the chemical treatment of London sewage 80 per cent of the suspended matter is removed.

Chemical Precipitation at Glasgow. The Glasgow plant is fully described by Moore & Silcock (1909). The superficial drainage area of Glasgow is about 39 square miles, extending along both sides of the Clyde, and the amount of sewage, with the proportion of rainfall which it is proposed to treat when this area is completely developed, is estimated to be 250,000,000 gallons per day. This sewage is to be treated at three separate plants. The first of these, built in 1894, is situated at Dalmarnock and treats at the present time 16,000,000 gallons of sewage daily. The second, built in 1904, is at Dalmuir, where the daily volume of sewage to be ultimately treated is 49,000,000 gallons per day. The third plant, being constructed at the present time, is at Shieldhall, where 47,000,000 gallons daily are to be treated.

The sewage at the Dalmarnock plant is run into two catch pits, 47 feet, 10 inches long, 20 feet broad and 10 feet deep, in which the heavier solids are deposited. The sewage is then pumped into a mixing pit, where the chemicals, lime and aluminium sulphate, are added in the proportion of 2 to 1. The amount used varies greatly, on account of the changing character of the sewage, from 4 to 31 grains lime per U.S. gallon, and from 2 to 15.5 grains aluminium sulphate. The sewage is passed

by continuous flow through precipitation tanks. The amount of sludge formed, 90 per cent moisture, averages about 40 long tons or 1400 cubic feet per million gallons.

The sludge, after the addition of lime, is pressed and sold to farmers. Part of the sludge after pressing is dried at a temperature of from 65 to 70° C., which reduces the water to 22.5 per cent; and it is sold under the name of "Globe Fertilizer."

The process used at Dalmuir, and to be used at Shieldhall, is similar to that at Dalmarnock, except that the sludge without being pressed is to be taken to sea by sludge steamers.

Chemical Precipitation in the United States. The two largest plants in the United States for the chemical treatment of sewage are at Worcester, Mass., and Providence, R. I.

At Worcester the plant is now used to treat that portion of the sewage which cannot be handled on intermittent filtration beds, the number of the latter not being sufficient to treat all the sewage. At Providence all of the sewage is treated, the effluent being discharged into Narragansett Bay. At both cities shallow tanks are used, and the sludge is pressed. At Worcester it is carried by cars to a deep isolated valley and dumped, little or no nuisance being created. At Providence it is carried by a double-bottom scow and dumped at the United States Government dumping station in 75 feet of water.

Worcester was the first large city in the United States which attempted to treat sewage before emptying it into a watercourse, and consequently the history of the plant, together with the present method of treatment, as furnished by Almon L. Fales, Chemist in charge of the plant, may be considered in some detail.

Treatment of Sewage at Worcester, Massachusetts. By an act of the state legislature in the year 1867, the city of Worcester, having a population of about 30,000, was granted the use of the various brooks within its limits for sewerage purposes. Accordingly, a sewer system was built upon the combined plan, emptying into the Blackstone River, a comparatively small

stream with a drainage area of about 60 square miles as it leaves the city.

As early as 1880 the towns along the river below Worcester began to complain of offensive odors. The outcome was an act of the legislature requiring the city to treat the sewage before discharging it into the stream.

A small chemical precipitation plant was put into operation in 1890 designed to treat 3,000,000 gallons of sewage daily. The works consisted chiefly of six settling basins, each having a capacity of 350,000 gallons, lime house, power plant and laboratory. In 1893 the number of settling basins was increased to sixteen, giving a total capacity of 5,500,000 gallons. The total cost of the precipitation works was approximately $200,000. This plant was capable of dealing with the entire dry weather flow of sewage and brook water, amounting to about 15,000,000 gallons daily, which was nearly the full capacity of the outfall sewer.

A sludge-pressing plant was built in 1898, and in the same year the city constructed fourteen acres of intermittent sand filtration beds. The number of beds has been increased from time to time, and at the close of 1909 there were 65.2 acres of actual filtering surface.

The sewerage system comprises 82 miles of separate sewers, 65 miles of combined sewers, these two systems being contributory to the disposal works, and 46 miles of surface water drains having no connection with the outfall sewer.

The dry weather flow of sewage is about 12,000,000 gallons and the average quantity treated amounts to about 15,000,000 gallons daily.

The sewage contains large amounts of trade waste, composed chiefly of pickling liquors from wire mills and foundries, and the refuse from tanneries. Its general character is shown by analyses given in Chapter I. At the present time two-thirds of the sewage is purified by chemical precipitation and one-third by intermittent filtration, this amount being controlled by the capacity of the filter beds.

All the sewage is passed through a grit chamber and that portion which is not to be purified on intermittent filtration beds, 10,000,000 gallons per day, is treated with lime, added in the form of milk of lime, and in sufficient quantity to render the

Fig. 29. Plan of Worcester Chemical Precipitation Plant.

sewage always slightly alkaline, the amounts required being from 900 to 1000 pounds per million gallons.

The ground plan of the precipitation plant is shown in Fig. 29. The milk of lime is added to the sewage at (R), about

100 feet above the mixing channel. The sewage passes through the mixing channel, in which are baffle boards, so as to insure thorough mixing of the lime with the sewage. The flow is then divided, a part passing to the right through basins, Nos. 1, 2 and 3, and the remainder passing through basins, Nos. 4, 5 and 6. These basins serve as roughing tanks, retaining the bulk of the sludge. The partially settled sewage then flows through the

FIG. 30. Bird's-eye view of Worcester Chemical Precipitation Plant.

finishing tanks, Nos. 7–16, in parallel, and finally into the channel (S) leading to the river. The flow through the tanks, averaging about 6 hours, is controlled by weirs across the outlets of the basins, and floating substances are prevented from passing off in the effluent by use of scum boards.

The sludge, containing 93 per cent water, averages 5000 gallons per million gallons of sewage. To remove this sludge from the basins, the water is drawn off by means of floating arms, and the sludge allowed to flow by gravity, each basin containing a central sludge channel, to the sump well (P). From the sump well it is pumped to the storage tank (MM) by means of a

Shone ejector, and 30–50 lbs. of lime in the form of milk of lime is added. The sludge is allowed to settle in the storage tanks and the clear liquid, 15 to 25 per cent of the total volume of the sludge, is drawn off on to sand filter beds.

The storage tanks are provided with vertical bar screens (L) with half-inch openings, and the screened sludge is pumped by power plunger pumps into the filter presses under 80 pounds' pressure per square inch.

FIG. 31. Sludge Presses at Worcester Chemical Precipitation Plant.

The liquor is conducted to sand filter beds, and the sludge cake, containing 70 per cent water, is hauled by motor cars to an isolated valley three quarters of a mile from the press house.

The removal of the suspended organic matter by the above process, measured by albuminoid ammonia, amounts to 85 to 90 per cent, which is about 50 per cent of the total organic mat-

ter in the sewage. The cost of the chemical treatment is about $10 per million gallons, a little more than half of which is to be charged to sludge disposal.

The sewage which is purified by intermittent filtration, 5 million gallons per day, after passing through the grit chamber is caused to flow through a sedimentation tank having a capacity of about one-half hour's flow. Here the grosser suspended matters are removed and disposed of by pumping onto adjacent fields.

The ordinary dose of sewage applied to the filters is from 300,000 to 400,000 gallons per bed, and the average rate per acre amounts to about 75,000 gallons daily. The accumulation on the surface of the beds is raked up late in the fall into small piles about 4 feet apart, which are left during the winter to assist in holding up the ice from the surface. In the spring the beds are given a thorough cleaning, and the surface accumulation, unavoidably mixed with some sand, is carted away and used for filling in the swamps and lowlands. The clogging matter so removed amounts to about 250 cubic yards per acre or 10 cubic yards per million gallons of sewage filtered. The cost of cleaning averages 35 cents per cubic yard.

The ferrous sulphate, present in great quantities in Worcester sewage, is largely oxidized during the process of filtration, and much of it settles out as hydrated ferric oxide in the gravel around the open joints of the underdrains, eventually clogging the openings. The drains in some of the beds have been relaid twice during the last ten years. This item increases the cost of operation about 20 per cent.

The total cost of operation of the filters amounts to about $275 per acre and $10 per million gallons of sewage filtered.

The entire cost of the disposal plant to date is $738,693.25. The net cost of maintenance for the fiscal year of 1909 amounted to $49,892.31, which is approximately 37 cents per capita.

Present Status of Chemical Precipitation. Though the chemical treatment of sewage gives an effluent much freer from suspended matter than either the effluent from simple sedimen-

tation or from the septic tank process, and consequently causes less trouble from the clogging of bacterial beds, the amount of sludge produced is very large, over fifty per cent more than the amount which accumulates in a sedimentation tank from the same sewage. This fact, with the cost of the chemicals used, has created a prejudice against chemical treatment, and in the United States practically no chemical precipitation plants have been built in recent years, and Fuller (1909) gives as his opinion the following:

"As a preparatory treatment for filtration its use has much more merit in the case of sewages highly charged with trade wastes than for ordinary domestic sewage. In some European projects chemical precipitation is still held to, because it is believed that its cost is justified by the increased rate at which the filters may be operated. With our dilute American sewage it is not believed that this will be the case under any ordinary circumstances."

There is, however, another side to this question, and during the past few years there has certainly been growing in England a feeling that for those cities which can dispose of the sludge by sending it in sewage tank steamers to be discharged into the ocean, it would be cheaper and better to remove all the suspended matter in sewage by adding chemicals than to allow the suspended matter contained in the effluent from plain sedimentation or septic tank treatment to accumulate upon bacterial beds; and it may be considered as debatable if even for certain inland cities the use of chemicals may not be advisable. In all processes of preliminary treatment adequate provisions must be made for the disposal of sludge, for the conclusions that were drawn in the early days of septic tank treatment, that the amount of sludge digested was so large that it would only be necessary to clean out the tank once in two or three years, have been greatly modified, and to-day it can be said that the average period between removals of sludge from septic tanks is often not over three months. Such being the case, the question of chemical treatment as a preliminary process resolves itself into deciding

whether or not in a given case the advantages of obtaining an effluent more easily and cheaply treated on bacterial beds than is otherwise obtainable, and with less danger of causing an aerial nuisance, do or do not offset the cost of the disposal of the large amount of sludge formed.

CHAPTER VI

PRELIMINARY TREATMENT OF SEWAGE BY THE SEPTIC PROCESS

The Nature of the Septic Process. Septic action may be popularly described as the decomposition, through bacterial agencies, of the sludge in the bottom of a sedimentation tank, with the consequent production of gases and the breaking up and partial liquefaction of the solid matter.

This action takes place whenever the sediment or sludge is not frequently removed from the tank. In hot summer weather it begins in a few days after sewage has been allowed to flow through a tank, and may reach its full development in less than a month; while in cold winter weather the bacterial action develops very much more slowly, often taking from two to three months. The decomposition of the organic matter is due chiefly to anaerobic bacteria, although undoubtedly worms and other animal forms play an important part.

It has been claimed that not only the suspended matter but also the organic matter in solution is partially decomposed, and, consequently, that the sewage after septic tank action is more easily and quickly oxidized on filter beds by the aerobic bacteria. This is questionable, and the general opinion at the present time is that the advantage of septic tank action lies merely in the reduction of the amount of sludge deposited by sedimentation.

The septic tank process must therefore be considered as a preliminary method of treatment, similar to sedimentation, the only difference being that in the septic tank process part of the suspended organic matter is liquefied with the evolution of gas. There is no question as to the liquefaction of the deposited matter in the septic tank, though the amount thus liquefied

varies very greatly at different places, and as a forcible illustration of this decomposition a description of Dunbar's experiments is well worth noting.

Dunbar says (Dunbar, 1908): "The author has investigated the subject by suspending in septic tanks a large number of solid organic substances, such as cooked vegetables, cabbages, turnips, potatoes, peas, beans, bread, various forms of cellulose, flesh in the form of the dead bodies of animals, skinned and unskinned, various kinds of fat, bones, cartilage, etc., and has shown that many of these substances are almost completely dissolved in from three to four weeks. They first presented a swollen appearance, and increased in weight. The turnips had holes on the surface, which gradually became deeper. The edges of the cabbage leaves looked as though they had been bitten, and similar signs of decomposition were visible in the case of the other substances. Of the skinned animals, the skeleton alone remained after a short time; with the unskinned animals the process lasted rather longer. At this stage I will only point out that the experiments were so arranged that no portion of the substances could be washed away; their disappearance was therefore due to solution and gasification. The skinned body of a guinea pig was allowed to remain in a septic tank for three weeks, when the clean white bones alone remained. . . . Objects suspended in the sludge itself decomposed almost as quickly as those suspended in the supernatant liquid." Calmette (1909) describes similar experiments in which cooked egg albumin was 99 per cent dissolved in 6 weeks. Of cooked meat 96 per cent disappeared in 6 weeks. Fish meat entirely disappeared in 2 weeks. Cabbage heads were reduced to 1 per cent of their original weight in 6 weeks. Fragments of meat which were almost wholly dissolved in 6 weeks when suspended in a septic tank, only lost 15 per cent of their weight in stagnant sewage from which the products of decomposition were not removed.

Early Development of the Septic Process. The essential processes of the septic tank are of course the same processes which have always taken place in the cesspool of our fore-

fathers; for the septic tank is simply a cesspool, regulated and controlled. The comprehension of the fundamental principles involved and the working out of the details of construction and operation best calculated to facilitate the process have gradually developed in the hands of various experimenters.

Leonard Metcalf, M. Am. Soc. C. E., in 1901 presented to the American Society of Civil Engineers a paper entitled "Antecedents of the Septic Tank." (Metcalf, 1901.) This paper and the appended discussion describe in an admirable manner the early applications of septic action, and include a number of diagramatic drawings of sewage tanks built previous to 1896. Certain of these are herewith reproduced in Figures 32–36; and the following historical notes are based largely on the data collected by Mr. Metcalf.

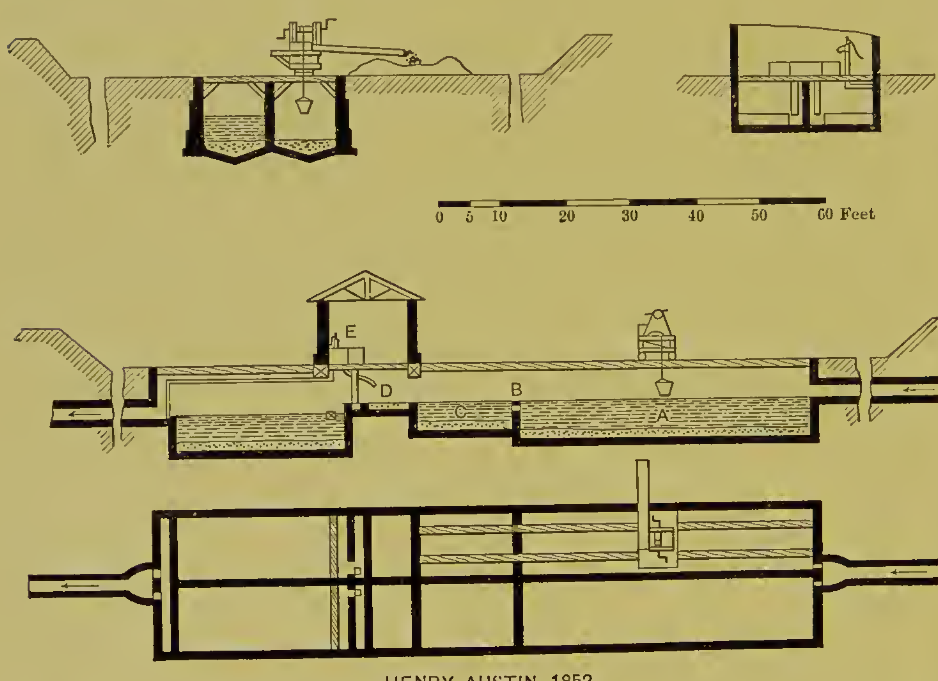


Fig. 32. The Austin Sewage Tank (copied by permission from Metcalf, 1901).

In 1857 Henry Austin, in a ' Report on Means for Deodorizing and Utilizing the Sewage of Towns," described a tank, designed by him, with a view to separating the solids from the

liquid before treating with lime (Fig. 32). This tank closely resembled modern tanks and included, from a practical standpoint, the essential elements of successful septic tank treatment as practiced to-day. The elements referred to are as follows:

"1. A deposit of solid matter on the bottom.

2. An accumulation of a solid floating mass on the surface.

3. The necessity for drawing off the liquid between the top and the bottom.

4. Provisions for a submerged nondisturbing outlet across the entire width of the tank below the top and above the bottom.

5. Desirability of drawing off upper layers of liquid from some of the compartments through strainers.

6. Placing of strainers at the end of the tank, forming thereby an outlet chamber, extending across the entire width of the tank and to any depth required.

7. Necessity for both a nondisturbing inlet and outlet in some tanks.

8. Use of baffle board or fenders at the inlet and a weir at the outlet end to accomplish nondisturbance and promote separation of the solids from the liquid sewage.

9. Provision for the removal of the accumulated solids from the tanks."

Probably the first definitely purposeful attempt to liquefy sludge was made by Louis H. Mouras of Vesoul, France, who designed the Automatic Scavenger, which in America would be called an overflow cesspool (Fig. 33). The Mouras Scavenger was "a closed vault with a water seal, which rapidly transforms all the excrementitious matter which it receives into a homogeneous fluid, only slightly turbid, and holding all the solid matters in suspension in the form of scarcely visible filaments." The action was attributed to anaerobic bacteria. This tank was introduced by Mouras about 1860, and was fully described by Abbé Moigno in the *Cosmos les Mondes* in 1881. Automatic scavengers of this type were widely used in Paris because a city ordinance then forbade the discharge of solid material into the sewers. They were patented in France, England, and the United

States in 1881–1882, twenty years after their first use. Although not entirely successful, and perhaps as used in Paris objectionable from the standpoint of the dissemination of odors and disease, these devices greatly reduced the amount of sludge to be handled.

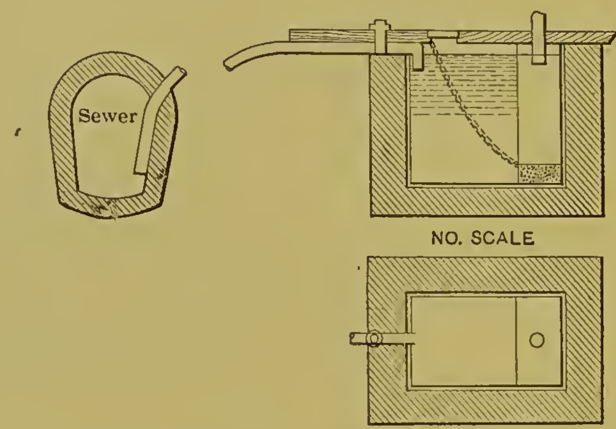


FIG. 33. The Mouras Scavenger (copied by permission from Metcalf, 1901).

Abbé Moigno, speaking of the liquefaction of the solid matter in the Mouras Automatic Scavenger, says:

"Daily observations conducted with a glass laboratory-scavenger have been made, and from these it results that fecal matters introduced on the 29th of August were entirely dissolved on the 16th of September. Even kitchen refuse, onion peelings, etc., which at first floated on the surface, descended after a time to the bottom of the vessel to await decomposition. Everything capable of being dissolved acted in a similar way, and even paper wholly disappeared."

Explaining this, he says, "The solvent action of sulphureted hydrogen is called into play, and a species of putrid fermentation is set up, which effects the liquefaction of the solid feces." Also, "May not the unseen agents be those vibrions or anaerobes which, according to Pasteur, are destroyed by oxygen, and only manifest their activity in vessels from which the air is excluded?"

The late Col. George E. Waring, Jr., in his book entitled "The Sanitary Drainage of Houses and Towns," published in

1876, describes a closed tank, built by him at Newport, in which "the solid deposit being organic matter decomposes in the form of ammonia which helps to dissolve the grease and make it soluble, so that both the deposit and the scum are constantly being washed away." In the next decade a number of tanks of this general type were built at various places in the United States. All embodied more or less clearly the principle of storage of sewage in a tank with submerged inlet and outlet. The designs installed at the Worcester (Mass.) Insane Hospital (1876), at Lawrenceville, (N. J.) (1882), and the Concord (Mass.) Reformatory (1883), and at Medfield, Mass. (1886), are well shown in Mr. Metcalf's figures. In 1883 the late Edw. S. Philbrick described a sewage disposal system designed by

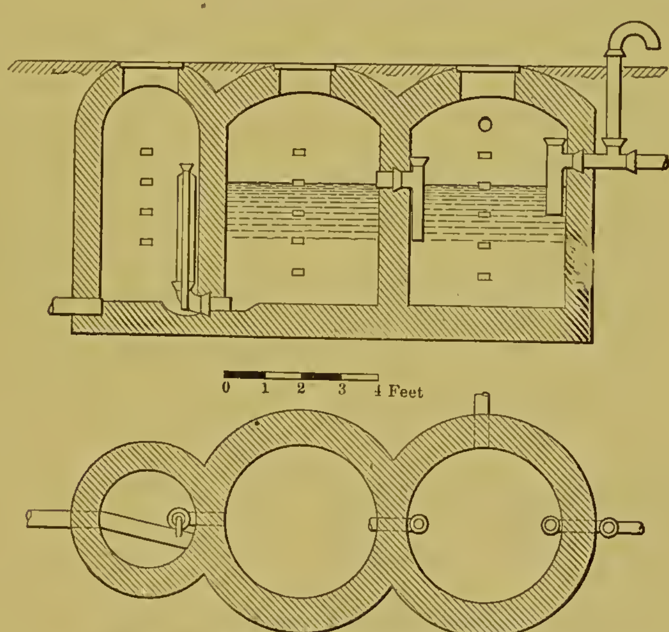


FIG. 34. Philbrick's Tank (copied by permission from Metcalf, 1901).

him in which there was provided "a tank or tank cesspool in which the solid particles of the sewage may become macerated and finely divided by fermentation before entering the distribution pipes" (see Fig. 34). Septic tanks were built in

Massachusetts at Gardner and Marlboro, in 1892, at Wellesley College in 1892 and at Amherst Agricultural College in 1893.

Mr. Scott-Moncrieff, the well-known sanitary engineer of Ashtead, England, was perhaps the first to point out clearly that from a bacteriological standpoint the purification of sewage took place in two stages, and that the first stage should serve as a prelude to further treatment. In 1891 he built at Ashtead a small plant in which the purification of the sewage took place in two stages. It consisted of a closed tank filled with stones, for the partial liquefaction of the solid matter in the sewage, and of open trays containing coke, for the second stage, where nitrification was to take place.

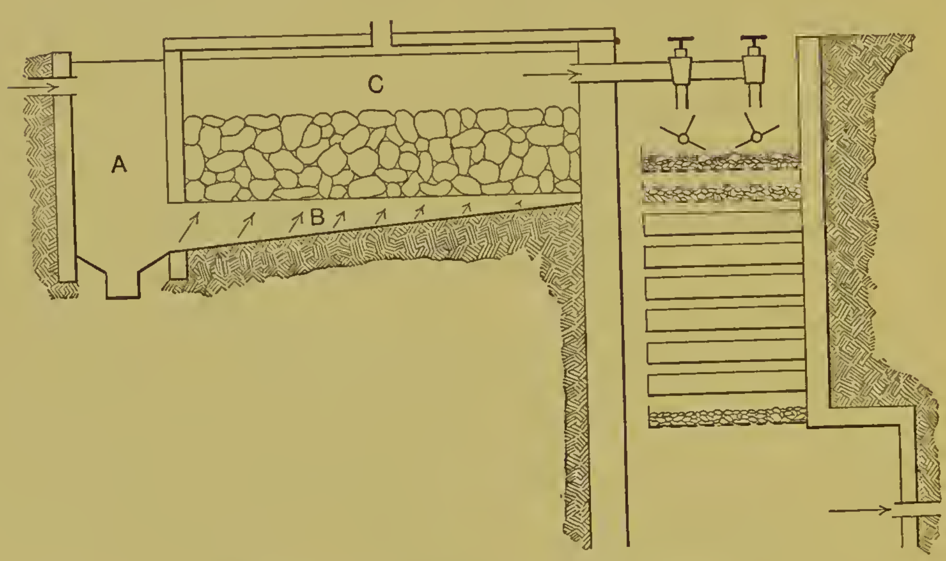


Fig. 35. Scott-Moncrieff's Tank and Filter.

Fig. 35 shows the liquefying tank and the coke trays used by Scott-Moncrieff. The sewage first entered the grease trap *A* and then passed into the space *B* of five cubic feet capacity, which was underneath the gratings of the liquefying tank *C*. It then passed upwards through a bed of small stones inside the tank, and then by means of distributors to the uppermost of a series of nine perforated trays, containing coke, supported vertically over one another, about three inches apart. Each tray had

an effective area of about one square foot, and contained seven inches of coke, broken to about one inch diameter. The time required for the liquid to pass through the nine trays was about ten minutes.

In the tank the anaerobic action took place, and on the coke in the trays, the aerobic. The anaerobic or septic action that went on in the liquefying tank when used at Ashtead was so effective that it is reported that the solid deposit of seven years from a household of ten persons was absorbed on nine square yards of land, causing no distinction in appearance between the soil and that surrounding it; and the second stage of purification (nitrification) was so perfect that all the decomposable matter was destroyed. In 1893 a plant on the same plan was designed by Scott-Moncrieff for the borough of Towchester. A year later, in

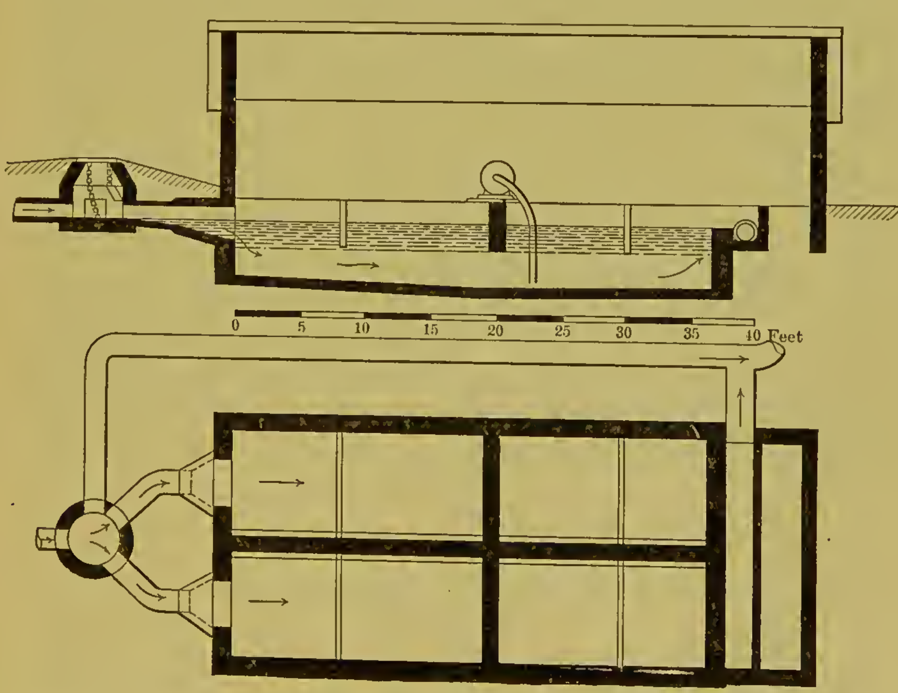


Fig. 36. Talbot's Champaign Tank (copied by permission from Metcalf, 1901).

1894, Prof. A. N. Talbot built a sewage tank at Urbana, Ill., in which the liquefying anaerobic action was observed; and a larger

plant, with this definite end in view, was designed for Champaign, Ill., in 1895 and built in 1897. (See Fig. 36.)

Cameron's Septic Tank. The anaerobic process of sewage purification owes its practical development chiefly to Donald Cameron, of Exeter, England, who holds much the same relation to this process that the Massachusetts State Board of Health holds toward intermittent filtration. In 1895 he installed a water-tight covered basin for the treatment of the sewage of a portion of the city by anaerobic putrefaction and gave it the picturesque name of the septic tank, by which it has since been known. The tank at Exeter (Fig. 37) was

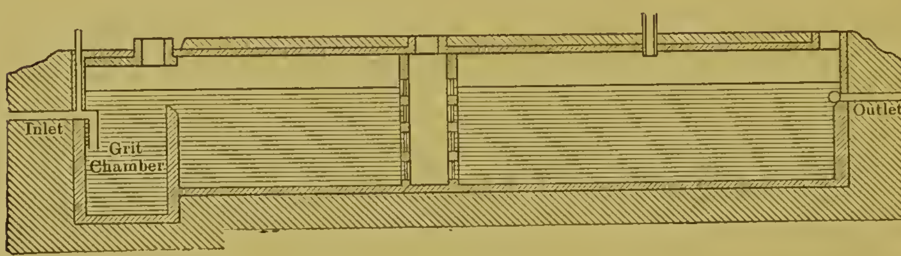


FIG. 37. Cameron's Septic Tank.

an underground tank of cement concrete, 65 feet long, 19 feet wide, and of an average depth of 7 feet, and having a capacity of 53,000 gallons. The tank was covered with a concrete arch, and a portion near the inlets was made about 3 feet deeper than the rest and partially cut off by a low wall, forming a couple of pockets or grit chambers, to retain sand, grit, and road washings. The inlet was carried down to a depth of 5 feet below the surface, so that air could not make its way down with the sewage, and also so that gases could not escape from the tank back into the sewer. The effluent outlet was also below the level of the liquid, and to avoid currents that might be liable to carry floating matter from the surface a cast-iron pipe was carried across the whole width of the tank 15 inches below the surface, and on the lower side of this pipe was a continuous opening about half an inch

in width. An iron pipe about one and a half inches in diameter extended up out of the top of the tank to allow the escape of gases, and the whole tank could be inspected from a central manhole provided with glass windows. In August, 1896, the main sewer of St. Leonard's, a suburb of Exeter, with a population of 1500, and an average daily flow of sewage of 57,000 gallons, was connected with the tank.

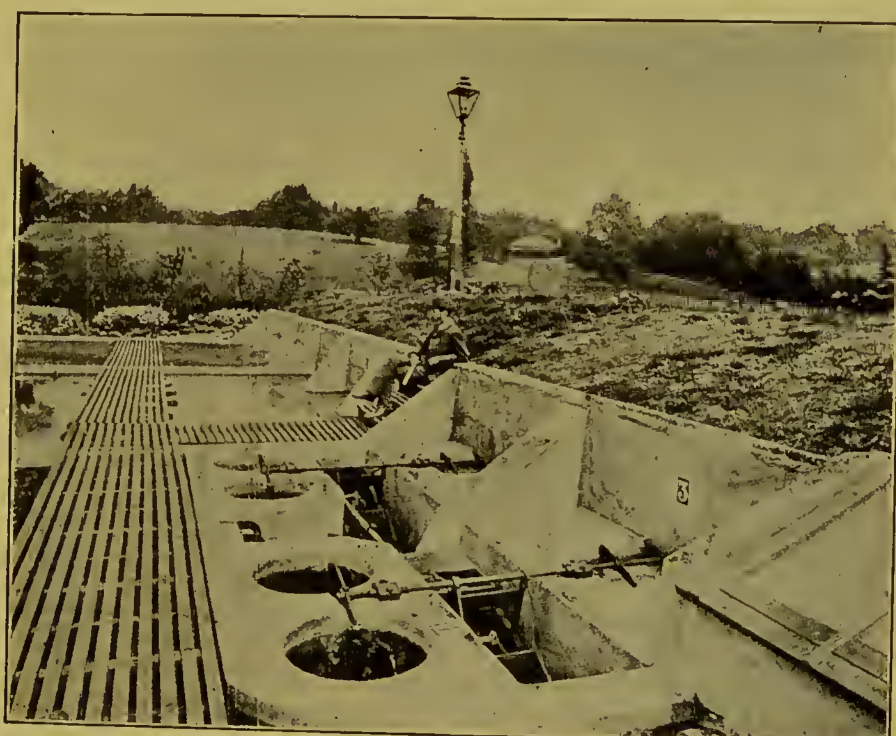


Fig. 38. View of Septic Tank and Contact Beds at Exeter, England.

The sewage at Exeter flowed slowly through the tank, taking about twenty-four hours in passage. The liquid turned dark-colored, while in the solids collected at the bottom an active fermentation was set up. Bubbles continually rose, carrying with them solid particles, which gathered at the surface to form a scum. At some plants this scum may be so firm and compact that a man could stand upon it. Meanwhile the effluent flowing

off was freed from gross floating matter, and its total solids and organic constituents were decreased to one-half and two-thirds their initial value, respectively. The material removed did not, however, merely accumulate in the tank, which was operated for three years without cleaning. At the end of the first year 25 tons of solids had been removed from the sewage, of which it was calculated that 5 tons remained in the tank, and this in the form of a rather stable peaty deposit, only one-third organic in composition. (Rideal, 1901.) These results attracted wide attention all over the world. Cameron's experiments were at once repeated and confirmed at Leeds and Manchester and elsewhere. In many places tanks which had been used for sedimentation and chemical precipitation were turned into septic tanks by merely changing the inlet and outlet so that the sewage could enter the tank, and the effluent leave the tank, 4 to 6 inches below the surface. Remarkable results were reported by some observers, who claimed that all of the organic matter was liquefied so that practically no sludge remained in the tank, and it was believed that at last the sludge disposal problem was solved. The earlier opinions regarding the liquefaction of sludge by this process have been very much modified, as will be shown later; but anaerobic preliminary treatment has won its place as one of the most valuable processes in the purification of sewage.

Chemistry and Bacteriology of the Septic Tank. The most characteristic processes which take place in the septic tank are hydrolyses, of which Rideal (1901) gives the following formula as an ideal type, taking $C_8H_{13}N_2O_3$ to represent the percentage composition of albumen:

$$4\ C_8H_{13}N_2O_3 + 14\ H_2O = 4\ N_2 + 19\ CH_4 + 13\ CO_2 + 2\ H_2$$

Other similar changes are effected without the addition of water; but all consist essentially in the decomposition of complex organic molecules into simpler compounds, without the addition of oxygen from external sources.

Rideal (1901) discusses the phenomena of anaerobic decomposition very thoroughly and classifies the important processes under eight main heads. 1. The first step is the hydrolysis of the complex albuminous bodies, which again includes two stages, —peptonization, or conversion into a soluble form, and splitting up of the resulting peptones into amido acids, leucin, tyrosin, etc., together with aromatic bodies. 2. The amido compounds formed under (1) are decomposed further into nitrogen or ammonia and fatty or aromatic acids. Whether nitrogen or ammonia shall be produced depends on the activity of the septic process and probably upon the types of bacteria present. Both reactions practically occur. In addition to the soluble or gaseous products there is a solid by-product which consists of a dark finely divided sediment, nitrogenous but stable, and resembling the peaty or humus matters in the soil. 3. In the breaking up of the original albuminoid molecule various organic acids are formed which break down to simpler acids and finally to carbonic acid and hydrogen or methane. Formates are decomposed to acid sodium carbonate, carbonic acid and hydrogen. With acetates the process is the same, except that methane takes the place of hydrogen. Lactates may break up in various ways to form propionic acid with valeric or acetic and succinic acids as by-products, or to form butyric acid with propionic acid or hydrogen as by-products. Malates may yield chiefly either propionic, succinic, butyric or lactic acids, with acetic or carbonic acids or hydrogen as by-products. Tartrates may form propionic, butyric or acetic acid, according to the type of fermentation, with alcohol and succinic acids as by-products in the latter case. Citrates produce chiefly acetic acid, and glycerates mainly acetic or formic acid. 4. The urea originally present in the sewage is directly hydrolized to form carbonic acid and ammonia. 5. Cellulose is decomposed to form fatty acids and carbonic acid and either methane or hydrogen, according to the type of micro-organism which is active. Omelianski (1906) records the quantitative results tabulated on page 120, after many weeks of action:

Under Type A the fatty acids consist mainly of acetic and butyric in the ratio 1.7 to 1.0. Under Type B about nine parts of acetic were found to one of butyric. 6. Starches, sugars

TABLE XXIV

END PRODUCTS OF CELLULOSE DECOMPOSITION

Grams (Omelianski, 1906.)

	Type A.	Type B.
Fatty acids	2.2402	1 0223
Carbonic acid	.9722	.8678
Undecomposed residue	.1272	.0750
Hydrogen	.0138	....
Methane	....	1372

and gums are quickly hydrolized and decomposed to lactic or butyric acids, carbonic acid, hydrogen and water. 7. The decomposition of fats is almost nil under anaerobic conditions. Glycerine is attacked and the fats emulsified, but the higher acids remain almost unchanged. Under aerobic conditions the fats themselves are acidified by certain species of bacteria, and still more actively by molds (Rahn, 1906). 8. A special set of fermentations liberates the sulphur of the organic molecule in the form of mercaptans or hydrogen sulphide. Most of the sulphur quickly combines with any iron present and is precipitated as finely divided sulphide.

The end results of the whole process are, then: (a) gases, including most of the H, N, CO_2 and CH_4, (b) substances in solution, including most of the ammonia, undecomposed amido bodies and fatty acids, (c) a solid residuum of stable peaty organic matter. The whole reaction is an exothermic one, evolving about 8 per cent as much heat energy as is left in the final products. (Rideal, 1901.)

The chief practical result of septic treatment is of course the reduction in suspended solids. Aside from this point, which will be discussed more fully later, the main analytical differences between the influent and the effluent of a septic tank lie in a decrease of albuminoid ammonia and oxygen consumed and an increase in free ammonia, the latter effect varying in different tanks.

TABLE XXV

ANALYTICAL RESULTS OF SEPTIC TREATMENT

Place.	Material.	Solids.		Nitrogen as —		Oxygen consumed.	Remarks.
		Total.	Suspended.	Free ammonia.	Albuminoid ammonia	Total.	
Exeter	Sewage	778	350	44.4	....	29.0	April to June, 1897.
	Tank effluent	593	154	32.5	....	20.1	
Leeds	Sewage	1.690	622	24.7	11.7	127.0	Feb., 1899, to Jan. 15, 1900.
	Tank effluent	1,090	183	21.0	5.2	58 5	
Birmingham	Sewage	1,967	676	31.9	13.7	153 0	Septic tank, No. 1, 1901, open tank.
	Tank effluent	1,399	245	43.3	18.7	108.0	
Lawrence	Sewage	769	232	38.1	7.0	49.5	Tank A, January, 1898, to January, 1903.
	Tank effluent	597	107	37.7	3.3	27 3	
Boston	Sewage	....	....	18.0	5.8	42.3	Average of 6 tanks, 1903–1905.
	Tank effluent	....	....	20.6	3.6	36.7	
Worcester	Sewage	832	311	20.2	8.4	110.7	1902, weekly samples.
	Tank effluent	625	214	25.8	5.6	70.7	

Note — Oxygen consumed in four hours at 80° F, in the English results: in two minutes boiling, at Lawrence, in ten minutes, at Worcester. Solids in Lawrence figures for year 1902 only.

The results of septic treatment in three English and three American cities are brought together in the table on page 121. In comparing them it will be noticed that the effect on free ammonia varies, this constituent sometimes decreasing appreciably, as at Exeter, but generally remaining fairly constant. Sometimes, as at Worcester, it exhibits a marked increase. The reactions in the septic tank naturally vary materially with the original composition and age of the sewage. In a very fresh sewage there is always a considerable formation of free ammonia by the decomposition of more complex organic bodies. If this process has been completed when the sewage is subjected to septic treatment a decrease in free ammonia may be expected in the tank. Albuminoid ammonia and oxygen consumed in each case fall to one-half or two-thirds of their initial value. The evidence accumulated by the Royal Sewage Commission indicated that an increase of free ammonia was the general rule in English septic tanks, while the albuminoid ammonia was reduced 38 to 54 per cent at Exeter, 50 per cent at Leicester, and 36 per cent at Birmingham. The oxygen consumed was reduced 25 to 33 per cent at Exeter, 50 per cent at Accrington, 50 per cent at Leeds, 36 to 60 per cent at Leicester, and 29 per cent at Birmingham. (Martin, 1905.)

The gas produced in the septic tank is a close measure of the amount of organic decomposition. Calmette (1909) calculates that a liter of methane represents either 1.7 grams of albumin or 2.4 grams of cellulose. The volume of gas was found by Fowler at Manchester (1901) to be 7.5 gallons per 100 gallons of sewage, and Clark (1900) at Lawrence obtained concordant results. Kinnicutt and Eddy (1902), on the other hand, found as an average, only about half this amount (3.9 gallons) produced by the septic treatment of the acid-iron sewage of Worcester. The composition of the gas also varies. The table below has been prepared from data given by Rideal, (1901) for Exeter, from Kinnicutt and Eddy's analyses of the gases from the experimental septic tank at Worcester, and from analyses of the gas from the Lawrence tank by A. H.

Gill. The Worcester results represent weekly analyses for a year.

At Worcester, during the warm summer months the amount of methane rose to 81 per cent and the carbon dioxide to 8.85,

TABLE XXVI

COMPOSITION OF SEPTIC TANK GASES

Per cent of various constituents

	Methane.	Nitrogen.	Carbon dioxide.	Hydrogen	Other gases.
Exeter	20.3	61.2	.3	18.2	....
Worcester	75.2	17.4	5.9	.3	1.4
Lawrence	78.9	16.3	3.4	....	....

the nitrogen falling to 8 per cent, and careful examinations of large volumes of gas failed to show the presence of either hydrogen sulphide or carbon monoxide.

Our knowledge of the bacteriology of the septic tank is somewhat fragmentary and unsatisfactory. What little is known of organic decomposition is well reviewed in Lafar's "Technical Mycology" by Miquel (1906), Hahn and Spieckermann (1906) and Omelianski (1906). According to Bienstock (1899), the initial decomposition of native proteids into aromatic oxy-acids, etc., depends upon a group of obligate anaerobes of which his *B. putrificus* is a type. Rettger (1908) and others have confirmed these results. On the other hand, it is strange that obligate anaerobes have never been found in any numbers either in feces (Bienstock, Rettger) or in sewage (Winslow and Belcher, 1904). Possibly symbiotic phenomena play an important part here; or the active organisms may be of types which cannot be cultivated on ordinary media. The later changes leading to the formation of hydrogen, carbon dioxide, methane and nitrogen are carried forward by a great many types of metatrophic bacteria, which can live either in the presence or the absence of oxygen. The simpler carbohydrates and organic acids are broken up by numerous species, *B. coli*, for example. The ammoniacal fermentation of urea may be due to various cocci and bacilli (Miquel, 1906). The two types of cellulose fermentation are carried

forward by specific anaerobic spore-forming rods (Omelianski, 1906). The sulphur fermentations have their own peculiar groups of organisms.

The bacteria, of course, effect all these changes by the secretion of enzymes, either within or without the bacterial cell. Much of the effect may be due to enzymes discharged from the cell and spread through the liquid contents of the tank.

It is somewhat misleading to speak of the action of the septic tank as due to "anaerobes" without qualification, for this is often taken to mean that strict anaerobes are the only active agents. This may perhaps be true of some stages in some of the processes (the first decomposition of proteids, for example). Most of the changes which occur are, however, carried out by facultative bacteria of various types. It is the conditions under which the organisms work which must be anaerobic in order that the process may go forward. Even the anaerobic condition can not be interpreted too broadly or too literally. Septic decomposition of sludge is not interfered with, and may perhaps even be favored by the presence of a slight amount of oxygen in the supernatant sewage. Recent investigations in Ohio have shown that of 19 tanks studied, the effluents of 8 contained dissolved oxygen ranging in quantity from 0.5 to 6 parts per million (Ohio, 1908).

One of the most striking characteristics of the septic process is the effect of temperature thereon. The amount of gas produced is a measure of septic activity. The following table, compiled by Fuller from Eddy and Kinnicutt's results, shows the greatly increased activity of the experimental septic tank at Worcester during the warmer months:

TABLE XXVII

RATIO OF THE VOLUME OF GAS PRODUCED EACH MONTH AT WORCESTER TO THE ANNUAL MEAN

January	30	July	140
February	62	August	167
March	48	September	170
April	51	October	116
May	100	November	115
June	148	December	65

The close relation between gas production and temperature is shown in Fig. 39, plotted, from the same Worcester data tabulated above, by Winslow and Phelps (1906). As a result of the temperature curve of septic action there is a great difference in the amount of sludge decomposed during different portions of the year. In winter sludge gradually accumulates, while in summer liquefaction exceeds deposition. Hence observations of the efficiency of a tank should cover all seasons of the year. It is important to remember that the periods of active fermentation are often marked by serious deterioration in the quality of the septic effluent on account of the fact that the

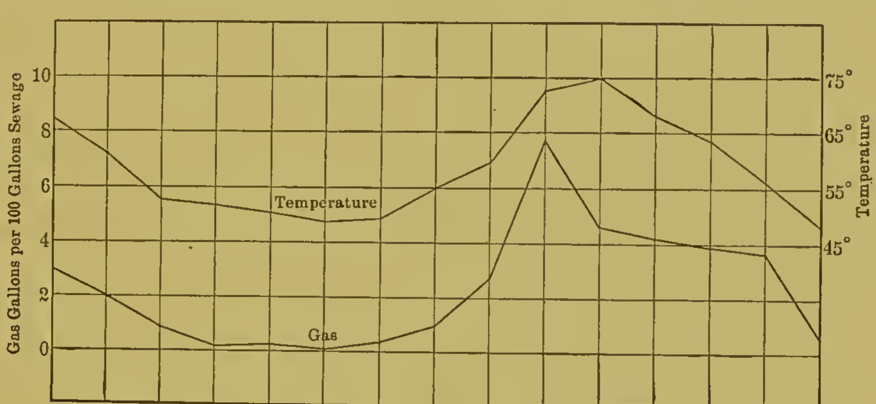


FIG. 39. Gas Production and Temperature, Worcester Septic Tank.

gas bubbles rising from the boiling sludge stir up the finer suspended material and carry it over to the outlet. The temperature factor has an important influence in controlling the varying success of the septic tank in different countries. An almost complete absence of liquefaction was reported in Russia from the results of Dzerszgowski's tank at Tsarskoé-Sélo. In India, on the other hand, septic tanks operate with marked success. A tank at Matunga was only emptied 3 times in 8 years and the percentage of volatile solids was reduced from 86 per cent in fresh sludge to 28 per cent in septic sludge (Calmette, 1909).

The course of septic action with a given sewage may frequently be profoundly modified by pecularities in the composition of the sewage itself. Thus the Worcester sewage, which contains 100

parts per million of free acidity, reckoned in terms of sulphuric acid, and 50–80 parts of iron in solution, yields, as has been pointed out, only 3.9 gallons of gas per 100 gallons of sewage, while results at Manchester and Lawrence show about double this amount. Furthermore the precipitation of iron sulphide has led to the presence of considerable fine suspended matter in the Worcester effluent (Kinnicutt and Eddy, 1903).

An interesting study of the delayed ripening of a septic tank has been made at Waterbury, Conn. (Taylor, 1909 *a*). Septic tanks started in September, 1905, showed no evidence whatever of septic action until May, 1906. After the latter date active ebullition set in and oxygen disappeared from the septic effluent.

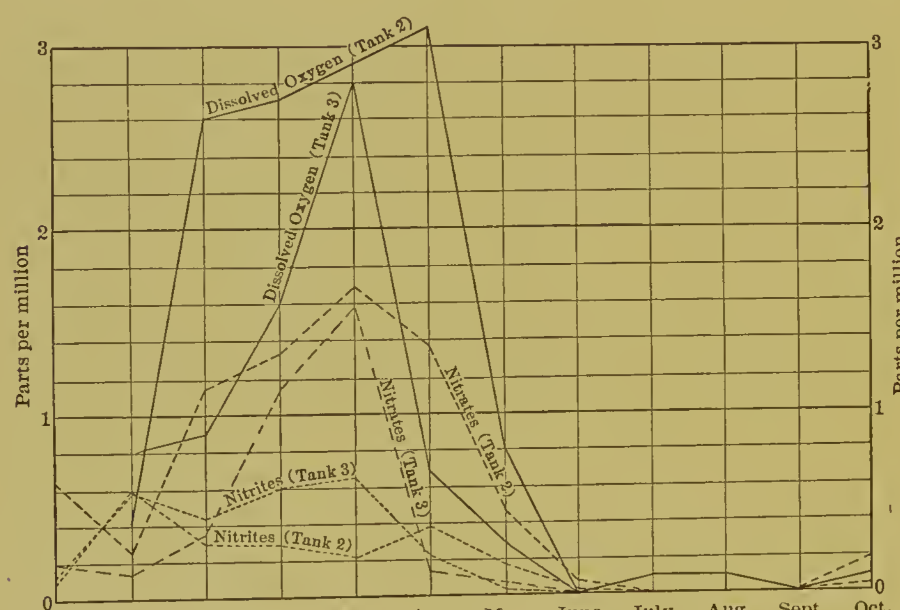


FIG. 40. Seasonal Variations in Septic Tank Effluent at Waterbury (Taylor, 1909 *a*)

The delayed onset of septic action is well shown by Mr. Taylor's curve (Fig. 40), and the table below, condensed from the figures given by him, indicates that the trouble lay in the large amount of oxygen present, during the colder months, in the applied sewage. Here there was too much oxygen for the inception of anaerobic changes on any appreciable scale.

TABLE XXVIII

SEASONAL COMPOSITION OF WATERBURY SEWAGE

(Taylor, 1909 *a*)

	Month, 1906.	Parts per Million.		
		Oxygen consumed.	Available oxygen as nitrates, nitrites and dissolved oxygen.	Ratio.
No septic action	January	9.2	10.8	1.17
	February	8.0	8.1	1.01
	March	8.8	11.7	1.33
	April	8.4	12.1	1.44
Septic action	May	8.8	8.4	.96
	June	8.4	6.0	.72
	July	8.6	4.7	.55
	August	9.0	3.9	.43
	September	10.4	4.7	.45
	October	11.8	8.6	.73

It may well be that in some cases inadequate septic action is due to the absence of specific types of bacteria essential to the process, rather than to chemical peculiarities in the composition of the sewage. The experiment has sometimes been tried of inoculating a septic tank which worked badly with old cesspool contents in the hope of remedying such defects; but the authors are not acquainted with any well-authenticated instance in which this has proved successful. At best the procedure seems a highly empirical one. What is really needed is definite scientific knowledge of the bacteriology of septic action. Only when such knowledge as this has been acquired can the process be rationally controlled.

Construction of Septic Tanks. A septic tank is of course essentially a sedimentation tank, with modifications, and the same broad rules of construction apply to both. What has been said of tanks in general in Chapter IV need not therefore be repeated here. The chief differences between the Cameron septic tank and the ordinary sedimentation tank lay in the fact that the former was covered, that it had a submerged inlet and outlet, that the flow was slower and, most important of all, that the sludge was allowed to remain in the tank to be decomposed instead of being removed at frequent intervals. The first of these

characteristics has now been generally abandoned except in cold climates and the last is a difference in operation only. The larger size and the arrangement of the inlet and outlet remain as the only peculiarities of construction which distinguish the septic tank from tanks of other forms.

The septic tank is always made relatively larger than the plain sedimentation tank, partly in order to allow for solution of fine suspended matter and partly in order to provide space for large accumulations of sludge without unduly increasing the rate of flow. This extra capacity may sometimes be provided for in the form of an additional unit which can be thrown in and out of service as desired. The lower limit of capacity is of course set by the fact that the period of storage must at least be long enough for the settling out of the maximum quantity of suspended solids. From laboratory experiments on sedimentation it appears that this maximum is about 80 per cent and that it will be nearly reached by six hours' storage (Steurnagel, 1904). In practice it has often been found that a longer period is of advantage, perhaps for the liquefaction of suspended solids of too fine a character to settle out. The Leeds results in the table below indicate an appreciably greater removal in twenty-four hours than in twelve hours, while further prolonging the period to forty-eight or seventy-two hours is of no advantage.

TABLE XXIX

AVERAGE OF ANALYSES ILLUSTRATING THE EFFECT OF DIFFERENT RATES OF FLOW THROUGH OPEN SEPTIC TANKS

(Leeds, 1905)

	12 hours' flow.		24 hours' flow.		48 hours' flow.		72 hours' flow.	
	Parts per million	Purification (per cent)	Parts per million	Purification (per cent)	Parts per million	Purification (per cent).	Parts per million.	Purification (per cent).
Total solids......	1,250	......	1,110	......	1,120	......	1,050	......
Suspended solids	272	52	162	71	155	73	141	76
Nitrogen as —								
Free ammonia	18.2	22	17.5	24	18.8	19	20.8	37
Albuminoid ammonia	6.3	50	5.2	58	4.5	64	4	52
Oxygen consumed in 4 hours at 80° F.	74 2	45	68 8	49	61 2	55	51 1	55

In making preliminary studies of large problems, where representative sewage may be obtained, experiments afford the most valuable data. Such studies were included in the Columbus experiments (Johnson, 1905), and showed that the most economical period of subsidence was longest when a strong sewage was being treated. With sewage diluted with storm (surface) water the suspended matter will, relatively speaking, settle out more quickly on account of its mineral character. With dilute night sewage, the rate of subsidence of the suspended matter is slow; but on account of the comparatively small initial quantity of suspended matter in such sewage, the amount remaining, after a given period, may be less than the amount remaining in a stronger sewage subjected to the same sedimentation. As a result of the Columbus experiments, there was recommended a sedimentation tank holding 8 hours' flow of sewage, to be operated on the septic plan.

Valuable experiments upon the "tanking" of sewage, with reference to the determination of proper shape and capacity, have been made by F. Wallis Stoddart (1905) at the Horfield sewage works at Bristol, England. These experiments were made with very strong domestic sewage, such as is frequently found in England; and the results are especially interesting when compared to the results of the Columbus experiments, which were made upon a typical American sewage discharged by a combined system of sewers. The first series of experiments at Bristol consisted in passing strong dry-weather sewage through the tank at constant rates for long periods. Tank capacities of 4 to 24 hours of flow were experimented with. As a result it was found that an 8-hour capacity gave the most satisfactory and economical results as regards the appearance of the tank effluent and subsequent treatment upon filters.

The second series of experiments at Bristol was planned to take account of the fluctuations in flow which occur in actual practice. With this series the sewage, instead of being passed through the tank at a uniform rate, was applied at varying rates, corresponding to about a 12-hour period flow in dry weather,

down to a 6-hour period, when the actual flow of sewage was increased correspondingly. That is, "the capacity of the tank being practically 5000 gallons, arrangements were made by which a portion of the crude sewage, corresponding in all respects as regards composition and rate of flow to the main supply, and varying in amount from 10,000 gallons per day in dry weather to 30,000 gallons when sufficient was entering the works, was admitted." The result of these experiments was to show that, where the flow is subjected to variations similar to those which occurred at Bristol, the tank should be designed for a 12-hour period, based on the average dry-weather flow. In this way the normal increase of about 50 per cent, which occurs during certain portions of the day, will change the rate of flow to correspond with the most favorable period (*i.e.* 8 hours), as determined by the first experiments; and in times of rain the tank will still be able to handle the sewage which comes to it.

The Royal Commission on Sewage Disposal makes the following statement in regard to the proper capacity for septic tanks:

"The rate of flow through a septic tank is consequently a matter in which the needs of each place require to be taken into account; but from general experience and a consideration of the evidence, we think it may safely be said that at few places should the sewage remain more than 24 hours or less than 12 in a septic tank."

This, in a general way, corresponds to the results obtained by Stoddart. Alvord (1902) and other American engineers provide shorter periods, often only four to eight hours, and some tanks operated on this principle, like that at Lake Forest, seem to work well. On the other hand, short periods of septic treatment at Wauwatosa and East Cleveland have yielded less satisfactory results (Winslow, 1905 *b*).

The following table gives the capacities of representative tanks, mostly operated on the septic plan, in use in Ohio:

TABLE XXX

Place.	Actual capacity of tank. Gallons.	Average linear velocity mm. per sec.	Hours flow	
			Average.	Minimum.
Delaware	100,000	.75	5.5	2.4
East Cleveland	170,000	.68	10.6	....
Geneva	39,000	.74	5.2	2.1
Kenton	18,800	.14	19.0	1.2
Lakewood	300,000	.50	12.4	1.8
London	34,700	.15	16.6	....
Mansfield	1,000,000	.32	24.0	....
Marion	414,000	.49	15.0	5.0
Westerville	22,000	.15	14.6	2.6
Sandusky — Soldiers' and Sailors' Home	109,000	.22	15.0	8.5

It is important that the septic period should not be too prolonged, since the anaerobic fermentation, if carried too far, may produce an effluent difficult to nitrify. Furthermore, it is probable that even the anaerobic action itself may be checked by the concentration of waste products with too long a period. An experiment at Lawrence is suggestive. A small septic tank was dosed, not with sewage, but with the more concentrated sludge from settled sewage. For six months the storage period was from five to fifteen days and sludge accumulated, filling up 60 per cent of the tank. The rate was then increased, so that the storage period was reduced to forty-nine hours, when the accumulated sludge decreased to 8 per cent and did not further increase for a year (Massachusetts, 1901). At Leeds it was found that a seventy-two-hour septic period interfered with the solution of sludge (Leeds, 1905). Clark and Gage (1905) have shown that certain types of bacteria especially active in sewage purification increase during the first twenty-four hours of septic treatment and then fall to numbers smaller than are present in raw sewage. It seems possible that too long a period of action may thus actually favor the accumulation of sludge while producing an effluent hard to nitrify. Alvord (1902) for these reasons suggests the use of an "elastic tank" with separate compartments, which can be included in or thrown out of the system to adjust it to varying conditions of flow and temperature.

The age of the sewage on reaching the plant is of course an important factor in designing a septic tank. A small plant treating fresh and undecomposed sewage, particularly if, as is usually the case, the hourly variations of flow are great, calls for a relatively larger tank. One day's flow might be considered a minimum capacity for such a plant, and two days' flow would probably prove more satisfactory.

The second important point in septic tank construction is the provision of submerged inlets and outlets so arranged that neither sludge nor scum may be unduly disturbed. The outlet of the original Cameron tank, as noted above, was a pipe running across the entire end of the tank 15 inches below the surface, with a slit in its under side. The outlets from the septic tanks at Mansfield, Ohio, are in the form of 98 2-inch pipes arranged in two rows at depths of 2 feet and 2 feet 6 inches below the flow line. At Saratoga, N. Y., each tank has two horizontal rows of similar 2-inch outlet pipes about 3.5 feet below the flow line.

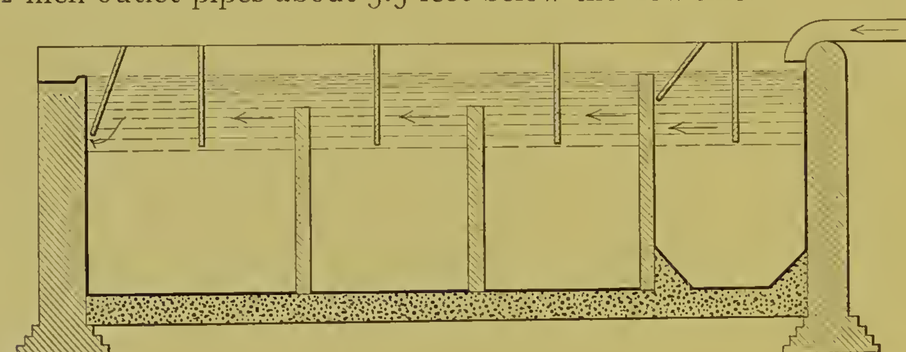


FIG. 41. Stoddart's Septic Tank.

An important though inexpensive feature of the design of all sedimentation tanks are the baffle walls and suspended baffles, or scum-boards, to prevent the forward movement of the deposited matter as well as the floating matter or scum, and, in addition, to cause a more thorough displacement as the sewage passes through the tank. Stoddart, after extended studies, decided that with his 5000-gallon experimental tank, operated on a 12-hour basis, the method of baffling shown in Fig. 41 was most efficient.

The first scum-board and baffle wall perform the chief function of holding back a large portion of the sludge and scum, while the remaining boards and baffle walls serve principally to distribute the flow evenly and to cause the sewage to take a longer course between inlet and outlet. The two inclined scum-boards have for their additional object the prevention of the escape of such suspended matter as may be violently upheaved from the bottom by gas arising from the putrefaction of the sludge.

Scum-boards may be attached to the walls in such a manner that they will be free to move vertically, and hence will rise and fall with the sewage. Instead of boards, iron plates, rigidly attached to the walls, are sometimes used. As both iron and wood, when used for this purpose, deteriorate more or less rapidly, it may be better to build the baffles of reinforced concrete in the form of deep, narrow beams, reaching across the tank.

The covering of septic tanks, although recommended by Cameron, has been found quite unnecessary for the maintenance of anaerobic conditions. If sewage be merely allowed to run slowly through an open tank, the general reactions appear to go on just the same. At Manchester the results from closed and open tanks under like conditions showed no marked difference, and in similar experiments at Leeds the open tank gave slightly better results, as shown below. For promoting anaerobic conditions tight covers are therefore needless.

TABLE XXXI

RESULTS FROM CLOSED AND OPEN SEPTIC TANKS AT LEEDS, ENGLAND

(Harrison, 1900.)

	Parts per Million.				
	Solids.		Nitrogen as —		
	Total.	Suspended.	Free ammonia.	Albuminoid ammonia.	Oxygen consumed in 4 hours at 80° F.
Open tank:					
Crude sewage	1,710	633	23.6	11.3	124.0
Effluent	1,110	172	20.6	4.9	54.3
Closed tank:					
Crude sewage	1,720	666	25.5	12.4	131.0
Effluent	1,130	197	20.0	5.0	69.3

The question of whether or not to cover a tank must be decided mainly in relation to its location with reference to habitations. If the tank is to be situated (say in the case of small plants) within 500 feet of dwellings, or if it is to be near a public highway or park, it would be safest to provide a roof, as there would doubtless be more or less odor emanating from the tank. Where the sewage is of such a nature that its retention in the tank causes the formation of a scum, then it is probable that this scum, if exposed to the weather, will produce odors, especially after it has been wet and the moisture is evaporating from its surface; and the mechanical disturbance of the scum caused by rain or wind will add to the odors.

Another point which should be considered in connection with providing a roof, is the winter temperature of the sewage taken in connection with the latitude of the place in which the tank is located. Where the water supply of the city is from surface sources the temperature of the sewage is likely to be very near the freezing point; but where the sewage is made up largely of ground water, the temperature will be somewhat higher. Thus, at Saratoga, N. Y., having a surface water supply, the sewage in the covered tanks has been known to freeze, whereas the sewage in places in Ohio, having ground water supplies, has not been known to freeze even when exposed in uncovered tanks.

In this connection it might be mentioned that roofs over tanks in which septic action is taking place should always be suitably ventilated in order to prevent explosive gases accumulating under them.

As to the kind of roof, there may be used either a wooden pitch roof or "housing," which should be high enough to permit ready access to, and inspection of, the contents of the tank, or a concrete roof provided with a sufficient number of manholes of generous size, to make the interior of the tank accessible and easily inspected. It would be desirable to have these manholes not more than 15 to 20 feet apart. Concrete roofs may be composed of reinforced beams and slabs, or they may be in the form of groined arches. The latter type is shown in Fig. 42 which

represents the tank at Mansfield, Ohio. It is the usual custom to cover the groined arch roof with one or two feet of earth, and in some cases this is done where the slab construction is used.

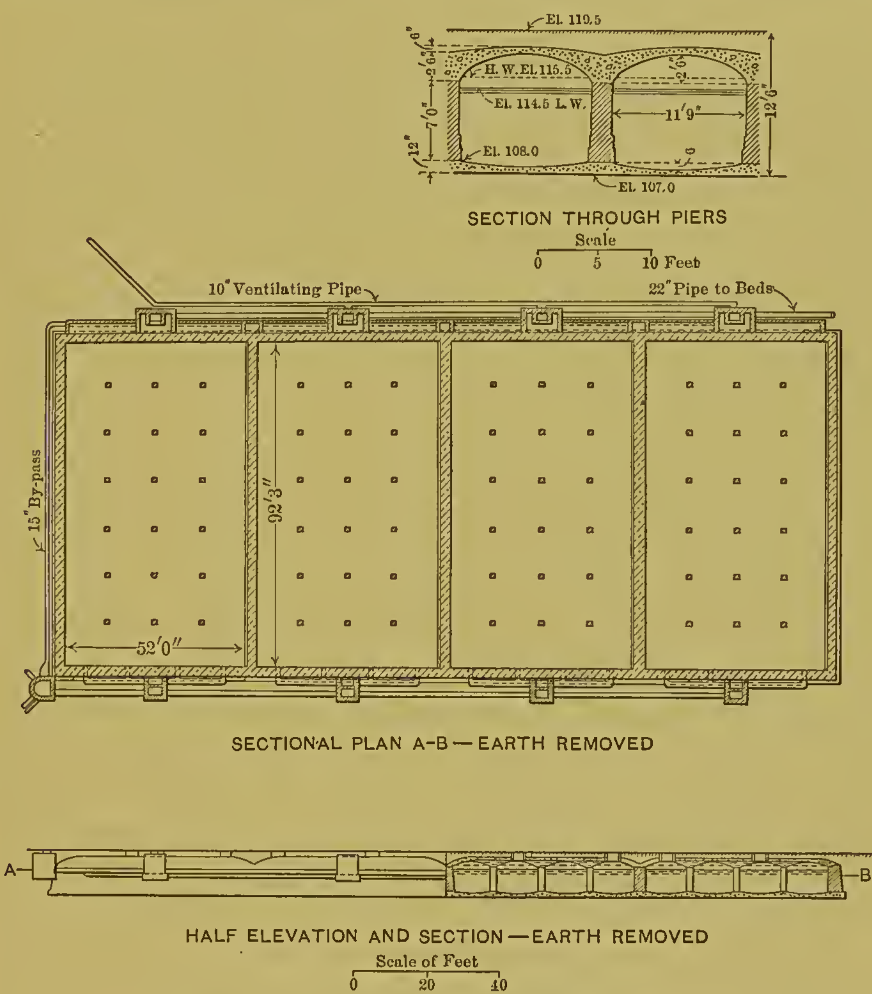


FIG. 42. Septic Tank at Mansfield, Ohio.

Septic tanks containing filling material of various kinds have been used in certain cases for increasing the removal of suspended solids by surface contacts. Scott-Moncrieff in his Ashtead experiments used, as already indicated, what was really a septic tank filled with stone. This principle has been applied in many other cases to the construction of anaerobic filters, lateral filters,

etc., of various types. The so-called "ladder filters" tested at Leeds, formed by a series of trays of stone, from one to the other of which the sewage flowed continuously, and operated on this principle, worked very badly. At Salford, roughing filters containing one-fourth inch to two-inch gravel were used for continuous filtration at a rate of 20 million gallons per acre per day. It was intended to wash these filters by upward flow with artificial aeration, but they have clogged seriously (Baker, 1904). At Lawrence a thorough study has been made of various strainers which operate with more or less continuous flow. All such devices, as well as the anaerobic filters installed at certain sewage plants in the Middle West, act like septic tanks, with the additional straining action due to the included material. Against this increased straining action must be set the tendency to clog and the difficulty of cleaning.

At many plants special devices or aerators are used for aerating the tank effluent. The purpose of this is to remove gases and products of bacterial decomposition, and if possible to cause the effluent to absorb oxygen, with the idea of making it more readily purified in the filters. It is debatable whether with effluents of ordinary concentration, and especially those already containing dissolved oxygen, such aeration is necessary. There is a distinct disadvantage, due to the liability of causing unnecessary odors, and there is also a loss in temperature in winter weather. On the other hand, with strong effluents having no dissolved oxygen, the process of aeration is considered desirable if not essential.

The Saratoga Tank. The tank at Saratoga, N. Y., built by F. A. Barbour, may be taken as one of the best types of successful septic tank construction in the United States (Barbour, 1905). The town is a famous pleasure resort with a normal population of 12,000, increased to 50,000 in midsummer. The daily flow of sewage varies correspondingly from 1,250,000 gallons a day to double this amount. The discharge of the crude sewage into brooks led to serious nuisances, and the town was forced to pay over $20,000 in damages; so in 1903 septic tanks and sand filters were installed. "The sewage, after screening, is pumped

to four septic tanks, each 91.5 feet long by 51.5 feet wide, having a total capacity of 1,000,000 gallons. The depth of sewage is 7.75 feet at the inlet and 8.25 feet at the outlet end.

"The entire structure is built of Portland cement concrete. The outside walls are 2 feet thick at the springing line of arches, vertical on the inside and with a batter of about $1\frac{1}{3}$ inches per foot on the outside. The division walls are 2 feet thick at the springing line and 3 feet thick at the level of the underside of floor. The piers are 18 inches square, the head being enlarged to 22 inches and the footing to 30 inches.

"The roof is of elliptical groined arch construction, the span being 11 feet 6 inches and the rise 2 feet 6 inches. The thickness at crown is 6 inches and the plane of extrados is depressed 9 inches over the piers. This depression is drained by a 2-inch pipe through the roof into the tanks.

"The floor is of inverted spherical groined arch construction, 6 inches thick at the center and 12 inches thick at the piers.

"The force main ends in a chamber, from which a pipe leads across the inlet ends of the tanks. This pipe is carried by a concrete bracket reinforced by old railroad iron. Inlet chambers permit the shutting off of one or more tanks as desired. A by-pass pipe leads from the chamber at the end of the force main around the tanks, so that raw sewage can be applied directly to the beds. Inside of the tanks the inlet pipe is split and carried across the end of tank on a concrete bracket, four openings being provided for the discharge of the sewage at an elevation 3.5 feet below the high-water line.

"The septic effluent escapes from the tanks through two horizontal rows of 2-inch pipe—ninety-six in all, set at an elevation about 3.5 feet below high-water line—into a narrow chamber extending the entire width of tank, from which it flows over a weir into the outlet chambers and thence to beds" (Barbour, 1905). A 24-inch sludge gate permits the emptying of the sludge onto the sludge beds located directly in front of the tanks, and 12-inch gates at a higher elevation make it possible to draw off the clear liquid between the scum and deposit and apply it to

FIG. 43. View of Interior of Septic Tank at Saratoga, N.Y. (courtesy of F. A. Barbour).

FIG. 44. Sewage Aerator at Saratoga, N.Y. (courtesy of F. A. Barbour).

any beds previous to the discharge of the sludge. The cost of the septic tanks was $15,500.

A special device, shown in Fig. 44, is used for aerating the septic effluent before it passes to the sand beds. The effluent flows over perforated sheet-iron plates, hung in three layers around a central riser pipe. The sewage entering the septic tank contained, in certain tests reported by Barbour, 4.3 per cent of

FIG. 45. Open Septic Tank at Columbus, Ohio (courtesy of J. H. Gregory).

the oxygen necessary for saturation. The septic effluent contained none; but after aeration the value rose to 70.4 per cent, falling again to 40.4 per cent before the effluent reached the beds. The rapid decrease after aeration shows the avidity for oxygen of the organic matter present and indicates that the process must materially facilitate the later work of nitrification.

An excellent example of open septic tank construction is shown in Fig. 45 which represents one of the secondary

tanks at Columbus, Ohio. The character of the walls and bottom baffles, the inlet pipes and sludge drains are all well illustrated.

Practical Results of Septic Treatment. The first point to consider in judging the efficiency of septic treatment is the reduction of suspended solids, measured by direct comparison of the applied sewage and the septic effluent. Tanks at Exeter, Leeds and Birmingham, as noted in the table on page 121, show a reduction of 56, 71 and 64 per cent, respectively. At Leicester the removal has ranged from 60 to 70 per cent. Experiments on London sewage at the Crossness outfall showed that six hours' sedimentation, in what was really a septic tank, gave a reduction from 281 to 125 parts of suspended solids (November, 1900, to March, 1901), and in another series (March to October, 1901) from 253 to 143 parts, a removal of 56 and 43 per cent, respectively. At Leeds 127 parts appeared in the septic effluent from March to June, 1899, 156 parts from July to October and 213 parts from November, 1899, to February, 1900 (Leeds, 1900). At Huddersfield the septic effluent contained 66 parts in August, 1900, 82 parts in September, 113 parts in October, 122 parts in November and 117 parts in December (R. S. C., 1902). These results illustrate the gradual increase in suspended solids discharged in the effluent, as the accumulation of sludge increases.

The table on page 141, from the final report of the British Royal Commission (R. S. C., 1908), gives a good idea of the results attained at a number of representative English plants.

Septic tanks in the United States show reductions of 65 per cent at Saratoga (Barbour, 1905), 61 per cent at Lawrence, Mass. (Mass., 1904), 40 per cent at Boston (Winslow and Phelps, 1907), and at Worcester 35 per cent (Kinnicutt and Eddy, 1903).

These are all average results. Many septic tanks in warm periods, when fermentation is active and gas bubbles are rising in great numbers, discharge a considerably greater amount of suspended solids. When gas ebullition is unusually violent, more suspended matter may leave the tank than enters it. This is strikingly illustrated by the table on page 142 showing results of

TABLE XXXII

Name of place.	Character of sewage.	Time of passage through tanks.	Per cent average reduction in suspended matter.	Suspended solids in tank liquor. Parts per million.	Remarks.
Accrington	Strong domestic sewage	Once in 42 hours	50	194	
Andover	Domestic, containing some brewery refuse.	Once in 19.3 hrs. (dry-weather flow) in 13.1 hrs. (max. flow).	30	110	8 mos. after incomplete sludging.
Caterham	Exceptionally strong domestic	Apparently 48 hrs. flow, when 3 out of the 4 hourly sets were drawn.	47	222	1 mo. after sludging.
Exeter (Main Works)	Strong domestic	Once in 11.5 hours (dry-weather flow).	66	125	18 mos. after sludging.
Exeter (St. Leonards)	Weak domestic	Once in 17.8 hours (dry-weather flow). Once in 8.6 hours (max. flow).	68	84	12 mos. after sludging.
Hartley Wintney	Domestic, with a considerable proportion of brewery refuse.	Once in 31.7 hours (dry-weather flow).	54	151	After $3\frac{1}{2}$ years without sludging.
Guildford	Strong domestic, containing a large proportion of brewery refuse.	Once in 36.7 hours	....	159	Tank full of sludge.
Knowle	Domestic	Once in 15.2 hours (dry-weather flow). Once in 3.2 hours (max. flow).	57	84	
Prestolee	Weak slop water sewage	Once in 22 hours (dry-weather flow).	42	32	After $4\frac{1}{2}$ years without sludging.
Rochdale	Strong manufacturing sewage, containing large quantity of wool scourings, etc.	Once in 30 hours	86	53	Tank in use for only 3 mos.
Slaithwaite	Dilute domestic (mainly slop-water).	Once in 13.6 hours (dry-weather flow).	34	71	After $5\frac{1}{2}$ years without sludging.
York	Weak domestic	Once in 26 hours	75	53	Two tanks, one in use 18 months and the other 6 months.

examinations of sewage tanks in Ohio (1908). It will be noted that most of the observations tabulated were made during the warmer months.

TABLE XXXIII

REMOVAL OF SUSPENDED SOLIDS IN OHIO SEPTIC TANKS

Place.	Date, 1906–'07.	Rate of sewage flow. Gals. in 24 hours.	Hours, flow.	Suspended Matter. Parts per million. Crude sewage.	Suspended Matter. Parts per million. Septic sewage.	Suspended Matter. Percentage removal.
Ashland	July 11–12, '06	150,000	6.3	70	50	29
Ashland	Apr. 24–25, '07	375,000	2.5	50	70	−40
East Cleveland	June 26–27, '06	365,000	11.2	210	400	−91
East Cleveland	July 11–12, '07	390,000	10.4	110	180	−64
Geneva	June 21–22, '06	181,000	5.2	60	65	− 8
Geneva	June 24–25, '07	204,000	4.6	90	150	−67
Kenton, N. District	October 5, '06	18,000	24.0	220	180	18
Kenton, N. District	July 2–3, '07	17,000	27.0	70	120	−72
Lakewood	June 12–14, '06	395,000	18.0	45	45	0
Lakewood	June 12–14, '07	1,150,000	6.2	100	140	−40
Mansfield	May 28–29, '07	1,058,000	23.0	75	110	−47
Marion	May 23–25, '06	415,000	24.0	150	85	43
Marion	Nov. 8–9, '06	370,000	27.0	90	140	−55
Marion	Apr. 9–11, '07	578,000	17.0	40	45	−12
Sandusky, Soldiers' and Sailors' Home	June 5–6, '06	155,000	17.0	95	90	5
Sandusky, Soldiers' and Sailors' Home	May 8–9, '07	174,000	15.0	95	90	5

Note. Suspended matter to nearest 5 parts below 100 and to nearest 10 parts above 100.

It is important to note in this connection that the sludge remaining in the septic tank is often much less offensive in nature than the sludge deposited from fresh sewage. Calmette (1909) found the differences between fresh sewage sludge and septic sludge to be quite marked, as indicated in the table below:

TABLE XXXIV

PERCENTAGE COMPOSITION OF FRESH AND SEPTIC SLUDGE

	Fresh sludge.	Septic sludge.
Volatile solids	45.8	32.6
Fixed solids	54.2	67.4
Nitrogen	2.0	1.3
Carbon	27.9	19.5
Fats	15.8	8.0

The action of the septic tank on dissolved solids is a variable one, as shown in the table below, compiled by Kinnicutt, with the addition of figures from a recent Lawrence report:

TABLE XXXV

REMOVAL OF SOLIDS BY THE SEPTIC TANK

(Kinnicutt, 1902; Clark, 1904.)

Place.	Solids removed (per cent of total).	
	Dissolved.	Suspended.
Exeter	−2.57	56.01
Lawrence	2.12	61.60
Leeds	12.05	70.37
Manchester	15.45	57.06
Worcester	20.67	25.57

It will be noticed that a slight removal of dissolved solids occurs, except at Exeter, reaching a considerable amount at Worcester. The phenomena in the case of Worcester are peculiar, on account of the acids and iron salts in the sewage. In the first place, all the reactions are hindered by the antiseptic action of these substances. The reduction of albuminoid ammonia is small, only 20 to 25 per cent; the gas production is only half that at Lawrence and Manchester; and the liquefaction of sludge is imperfect. In the second place, the proportionate decrease of suspended solids is small and that of dissolved solids great, on account of the reduction and precipitation of iron as sulphide.

In addition to the reduction in suspended solids, it is claimed by some experts that the anaerobic putrefaction brings the soluble constituents into a form in which they are more easily acted on by the nitrifying organisms. Martin, Cameron, and Fowler all expressed this opinion before the Royal Sewage Commission (Martin, 1905). The writers are not aware of any data which support this contention. Harding and Frankland (Martin, 1905) are skeptical as to such an advantage and Dibdin (1904) wholly disbelieves it. On the other hand, it is probable, as was shown before the Royal Sewage Commission, that when

not accurately regulated, " the anaerobic process may be carried too far, so as to interfere with the subsequent aerobic action " (Dibdin, 1903). Martin and Rideal minimize such interference, while Scott-Moncrieff, Woodhead and Fowler consider it of great importance (Martin, 1905). It appears certain that with strong sewage the putrefactive process may be carried so far that its products will check the aerobic organisms. In experiments at Caterham an effluent was obtained containing 1260 parts per million of dissolved solids, 288 parts of nitrogen as free ammonia and 54 parts of organic nitrogen, which would not undergo nitrification until diluted (Rideal, 1901). Experience at Andover leads to the same conclusion. Here the sewage is strong and already twenty-four hours old when it reaches the disposal area. Most of it is discharged on sand beds without further treatment. While the beds were successfully handling raw sewage at a rate of 30,000 gallons per acre per day, a small filter gave poor results with septic effluent at a rate of 40,000 and very bad results when the rate was increased to 100,000 (Clark, 1900).

In support of the belief that septic sewage cannot be efficiently treated in oxidizing filters, Dunbar (1908) states that at Hamburg contact beds could be filled six times a day with fresh sewage without yielding an unsatisfactory effluent, whereas they would only take septic sewage twice a day. It is believed that contrary opinions held by certain authors were due to the fact that septic tank effluent was compared to unsettled and unclarified fresh sewage, which of course contained clogging material.

The experience gained in the study of Ohio plants (Ohio, 1908) suggests that when treating weak American sewage in moderate-sized tanks, little fear need be entertained of rendering the effluent so septic as to be less responsive to treatment in filters.

The opinion that septic action destroys pathogenic bacteria has been occasionally expressed by various observers. This is true only to a limited degree, and no reliance should be placed upon such an action where sewage is to be purified with a view to protecting a water supply. Of course any process which removes the suspended matter from sewage, removes to a cor-

responding degree the bacteria which are attached to the solid particles. The claim that septic action destroys pathogenic bacteria to a greater extent than that just mentioned is, however, based on certain theories of bacterial antagonism which have not yet been placed on a definite basis. Furthermore, most of the experimental evidence has indicated that such bacterial antagonism, if it exists at all, cannot be depended upon to any considerable extent.

The second main criterion for judging the efficiency of a septic tank depends on the liquefaction of the suspended solids. Granting that the tank effects a removal of 60 to 70 per cent of suspended solids under favorable conditions, the fate of the matter retained must next be determined — how much is stored as sludge and how much is reduced to liquid or gaseous form. Evidence before the English Royal Commission indicates widely varying results with different tanks. Watson at Birmingham, after four years of careful study, believes that the digestion of sludge is not over 10 per cent. On the other hand, the following results have been reported at other English towns: a reduction amounting to 26 per cent at Manchester, 20 to 60 per cent at Leeds, 30 per cent at Sheffield, 35 per cent at Accrington, 40 per cent at Huddersfield, 50 per cent at Glasgow, and 80 per cent at Exeter (Martin, 1905). In the London experiments the destruction of total sludge was 41 per cent and of organic sludge 71 per cent (Dibdin, 1903). At Hampton 58 per cent of the organic sludge was destroyed (Baker, 1904). Careful studies made by the Royal Commission, and extending over a two-year period, showed a digestion of 25 per cent at Exeter and 30 per cent at Ilford (R. S. C., 1908). It is rather interesting to note that the original reports from Exeter showed a digestion of 80 per cent against only 25 per cent observed by the Royal Commission.

In the United States the Saratoga tank has been one of the most successful in its operation, perhaps on account of the fact that it treats purely domestic sewage and also because its heaviest burden comes in summer when the septic processes are most vigorous. In the first two years of its operation it received about

1,000,000 pounds of suspended solids and discharged in the effluent, 350,000 pounds. Of the 650,000 pounds remaining in the tank only 200,000 were stored, a decomposition of 450,000 pounds or 69 per cent of the solids remaining in the tank. The experimental tanks of the Technology Experiment Station at Boston received 3790 pounds of suspended solids in the two years, 1905–07; 1120 pounds were retained by the tank, of which 675 pounds were stored and 471 pounds, or 42 per cent, liquefied. The experimental tank at Worcester removed from the sewage from July, 1900, to Oct., 1902, 1193 lbs. of suspended matter, and in Oct., 1902, contained 729 lbs. in the sludge, giving as the amount liquefied, 39 per cent.

As seems evident from these various analytical results, the amount of sludge digestion in the septic tank varies so widely in different places that it is difficult to make any clear generalizations. The proportion of organic and inorganic material in the sludge must of course markedly affect the results, since only the former can be decomposed. The composition of the sewage is also undoubtedly of much importance; it may well be that the poor results reported from Birmingham are due to the presence of harmful manufactural wastes.

The frequency with which septic tank sludge must be removed naturally varies widely, according to the success of the liquefying process. It is desirable to postpone cleaning as long as possible, on account of the progressive consolidation of the sludge and its gradual decrease in putrescibility. According to the Royal Commission studies, the water content of fresh sludge is 90–95 per cent, while after storage this value may be reduced to 80 or 85 per cent. A decrease of water content from 95 per cent to 80 per cent means only one-fourth the volume of sludge to handle. When, however, the sludge occupies a third of the tank capacity or more, suspended matter is pretty sure to begin to come over in the effluent, and the tank must be cleaned.

Practical experience in regard to the necessity of cleaning septic tanks has varied within wide limits. At Exeter Cameron's original tank was operated for eight years without cleaning, but from

the present installation, with a fourteen-hour period, sludge is pumped out once a month. Baker reported, from a study of English plants in 1904, that at Barrhead a tank had been in use for six years (twenty-four-hour period) without cleaning and with little deposit. At Acton (sixteen hours) a tank had been operated for fifteen months with no deposit. At Yeovil (twenty-four hours), Burnley (twelve hours), Sutton (five hours) and Accrington (twenty-eight hours) it had been found necessary to remove sludge about once a year. At Oldham the tank was cleaned every two or three months (Baker, 1904). American plants exhibit similar variations. The Mansfield tank was not cleaned for four years after it was installed; but its success in this regard is largely due to the fact that much of the suspended matter entering the tank passes on through it and is not removed at all (Ohio, 1908). At Plainfield, N. J., the septic tanks have caused much trouble. At Saratoga, on the other hand, sludge was not removed for five years.

One of the most striking differences between individual septic tanks lies in the presence or absence of a surface scum. Scum is intimately related to the amount of gas evolved and to the character of the suspended matter in the sewage. The material composing the scum is carried from bottom to surface by the gas, as it escapes. The material tends at first to sink again, but becomes matted together by means of paper, hair, fat, etc., as well as by vegetable molds, with a result that a tough floating mass is formed which, in some instances, is strong enough to bear the weight of a man. Grass and vegetation sometimes flourish on top of the scum. Below is an analysis of scum from a septic tank at Worcester:

TABLE XXXVI

	Per cent.
Silica, SiO_2	12.62
Iron sulphide, FeS	8.93
Iron, Fe (not united to sulphur)	1.61
Sulphur, S, not as sulphide	0.00
Alumina, Al_2O_3	12.62
Lime, CaO	4.23
Carbon, C	41.83
Hydrogen, H	6.51
Nitrogen, N	5.20

It was formerly believed that scum was essential to septic action — it has been shown, however, that scum is only incidental to such action, and indeed may be distinctly detrimental to it, since the solids in the upper layers of the scum are removed from the sphere of most efficient septic decomposition. The scum is thickest in tanks receiving unscreened sewage, and particularly if the sewage contains a large proportion of street washings. The grain from horse droppings furnishes an excellent basis for the growth of scum. In a given tank, the thickness of the

FIG. 46. Scum on Septic Tank at Worcester, Mass.

scum varies with the seasons, and with the bacterial activity, although two tanks located in the same climate may have scums differing widely in thickness. At Washington, Pa., the scum at the upper end of a tank receiving unscreened sewage became so thick as to occupy the entire depth. When tanks receive well-screened sewage, or very dilute sewage, little or no scum is formed. This was the case with the experimental tanks at Columbus. Rain falling upon scum in an open tank will tend

to break it up and may cause odors by moistening the usually dry surface.

Starting with a clean tank, the time required to establish septic action varies, as does the intensity of the action, with the temperature, the action of course, beginning sooner in the summer months. The presence of certain kinds of bacteria is probably necessary to start and to carry on septic action. These bacteria, under ordinary conditions, exist in the sewage and soon establish themselves in the tank. In certain cases, however, as discussed above, these bacteria may be destroyed by acid contained in the sewage, as at Shelby, Ohio (Ohio, 1908). It is believed that the action may be more quickly brought about by "seeding" the clean tank with sludge from an active one.

Definite figures, as to the time required for the natural ripening of a tank cannot be given. Frequently one or two weeks will suffice, while on the other hand, in cold weather or with a sewage of unfavorable character, three or four months might be necessary. As will be discussed later in this chapter, the Court, in the case of the Cameron Septic Tank Co. *vs.* Saratoga Springs, considered that six weeks was a reasonable average estimate of the time required to establish septic action.

Obnoxious odors may arise from septic tanks or from the effluent passing out of the tank. With small installations, very little odor, as a rule, is noticed. As mentioned above, odors when present, often come from the moist scum rather than from the sewage itself. In England, aerial nuisances from many septic tanks have been the cause of complaint, yet at Birmingham, the largest septic tank installation in the world, odors are practically absent, possibly as a result of the character of the sewage.

The consulting engineers for the new plant at Baltimore, Md., which is to treat the sewage of 600,000 people, concluded that it was safe to predict that there need be no objectionable odors more than one-fourth mile from the tanks.

Septic Tank Patent Litigation. The subject of patent rights relating to septic action is an interesting and important one, especially in the United States. Litigation on the matter has

made many local officials afraid to adopt plans contemplating the use of sewage tanks, and has thus tended to cause delays in the carrying on of public improvements; and it has perhaps given undue prominence to the importance of septic treatment, which is, at best, simply one of several feasible methods of preliminary treatment, in its relation to the broader problems of sewage disposal.

It has been pointed out above that the septic principle has long been utilized in England, in France and in the United States for the sedimentation and decomposition of sewage sludge; yet it is certain that the extensive experiments of Donald Cameron had great influence in showing that the principles of bacterial decomposition could in many cases be utilized to better advantage, and on a larger scale, than had hitherto been generally realized. The experiments are fully described in the "Interim Report of the Royal Commission on Sewage Disposal," Vol. II, published in 1902. Their practical importance in the history of the process is well stated as follows, in the final report of the Royal Commission (R. S. C., 1908): "The notion that the solid matter of sewage would be digested by passing the sewage through a sealed tank is by no means novel, but it does not appear to have had any extensive practical application until Mr. Cameron, who held the office of City Surveyor of Exeter, proposed the adoption of the 'septic tank treatment' for that city."

On November 8, 1895, Cameron applied for a patent, and on April 25, 1896, there was granted to him British patent No. 21,142, covering the process of liquefying, by bacterial action, sludge deposited in a flowing stream of sewage; it also covered certain constructional details of design. Although rights under this patent have never been enforced in England, the granting of the British patent led to the issuance of the United States patent, which has been the basis of interesting litigation and discussion in this country.

United States patent No. 634,423 was issued to Cameron, Commin & Martin on October 3rd, 1899, and unless extended, expired November 2nd, 1909. This patent soon after became the prop-

erty of The Cameron Septic Tank Co. of Chicago, Illinois. It contains 22 claims, which may be divided into two classes: the process claims and the apparatus claims. The significant part of the process is stated in the following words, modified in other claims to include the phenomenon of scum formation and to specify the subsequent treatment of the effluent by aerobic filtration:

"The process of liquefying the solid matter contained in sewage, which consists in secluding a pool of sewage having a non-disturbing inflow and outflow, from light, air and agitation until a mass of micro-organisms has been developed of a character and quantity sufficient to liquefy the solid matter of the flowing sewage, the inflow serving to sustain the micro-organisms, and then subjecting said pool under exclusion of light and air and under a non-disturbing inflow and outflow to the liquefying action of the so-cultivated micro-organisms until the solid organic matter contained in the flowing sewage is dissolved." The process claims cover a submerged outlet extending across the greater part of the width of the tank, a non-disturbing inlet, and the exclusion of light and air.

The first lawsuit of importance concerning the validity of the patent was that begun in 1904, of "The Cameron Septic Tank Co., Complainant, *vs.* The Village of Saratoga Springs and the Sewer, Water and Streets Commission of Saratoga Springs." The suit was brought because the village had installed septic tanks, aerating device and intermittent sand filters according to plans drawn by Snow & Barbour, to which reference has been made above. The court record of the case, which is printed in two volumes, contains the evidence of a large number of prominent experts, including a most interesting review not only of the patent claim in question, but also of the many phases of the prior art of sewage treatment.

In March, 1907, the United States Circuit Court, Northern District of New York, awarded judgment to the defendants (i.e., in favor of Saratoga Springs) and dismissed the suit. The trial judge, Judge Ray, in his opinion, says:

"The Cameron process, if it be a process, consists in constructing a tank in a certain way in which to enable a process of nature to be performed, and as he has borrowed the tank, the inlet, and, in the main, the outlet from the prior art, changing or improving no element except it be the outlet apparatus, and produces no new result, the defendant does not infringe in using the old tank, and inlet with an outlet entirely different from Cameron to produce the same result. In fact, the inlet, and outlet, and aerator, and filter, all are essentially different from Cameron. But I prefer to base my decision on the broad ground that the claims in suit of the Cameron patent, in view of the prior art, are invalid for want of patentable invention. Cameron may have carried forward discovery in this art, and may have improved means, but as defendants' process and apparatus are clearly differentiated there is no infringement."

The case was carried up, however, to the United States Court of Appeals for the Second Circuit, with the result that (on January 7th, 1908) the decision of the lower court was partially reversed, and the Saratoga tanks were found to infringe claims Nos. 1, 2, 3, 4 and 21, relating to the *process* of septic action. The remaining seventeen claims of the patent were, however, found invalid, as in the decision of the lower court. The opinion of the higher court, in upholding the "Process Claims" read in part as follows:

"We, however, are satisfied that Cameron was the first one to subject a flowing current of sewage to the action of anaerobes and aerobes under conditions which secured their separate and successive action, the action of the segregated anaerobes fitting the effluent for subsequent filtration and aerobic action."

The opinion of the Court of Appeals came as a surprise to many; and with a view to carrying the case, on behalf of Saratoga, to the United States Supreme Court, and thus gaining opportunity to present additional evidence on the practical workings of septic treatment, a number of interested engineers, and others, organized in March, 1908, the "Association for the Defense of Septic Process Suits." The Supreme Court refused to hear the case.

The Association, however, effected a permanent organization for the purpose of assisting in the adjustment of royalties and for the mutual defense of such suits as may be brought against owners of tanks which do not infringe, or which are located outside the Second Circuit. Up to the time of writing, no decision has been reached as to the amount of damages to be paid by the village of Saratoga Springs. The defendants contend that such damages should be no more than the cost of cleaning the tanks once every six weeks since operation began, as the trial records indicated that such cleaning prevents septic action, and hence avoids infringement.

Hydrolytic Tanks and the Hampton Doctrine. One of the most interesting modifications of tank treatment for the removal of solids from sewage is the Hydrolytic process, developed at Hampton, England, which has not only advanced the practical art of sewage treatment but has contributed in an important degree to a just conception of its theoretical principles. The most interesting features of the Hampton process have nothing to do with septic action, being, in fact, purely physical, rather than chemical or bacteriological. Nevertheless the hydrolytic tank was historically an outgrowth of the septic tank and may therefore for convenience be considered in this chapter. As a matter of fact, any division line between the processes of sewage purification must be somewhat arbitrary. The septic tank, the hydrolytic tank, the Dibdin plate bed and the contact bed really make up an intergrading series, although the first two are primarily anaerobic and the last primarily aerobic.

The first step in the development of the Hampton process was the separation of the septic tank into compartments,—two lateral ones, through which the fresh sewage flowed, and a third, between and below the others, into which the sludge settled through special openings and in which it remained until removed. The general arrangement of the original Hampton tanks is shown in Fig. 47. The sewage flowed through compartments shown at *A* and the sludge settled through the openings *B* into the liquefying chamber *C*. The object of providing the separate

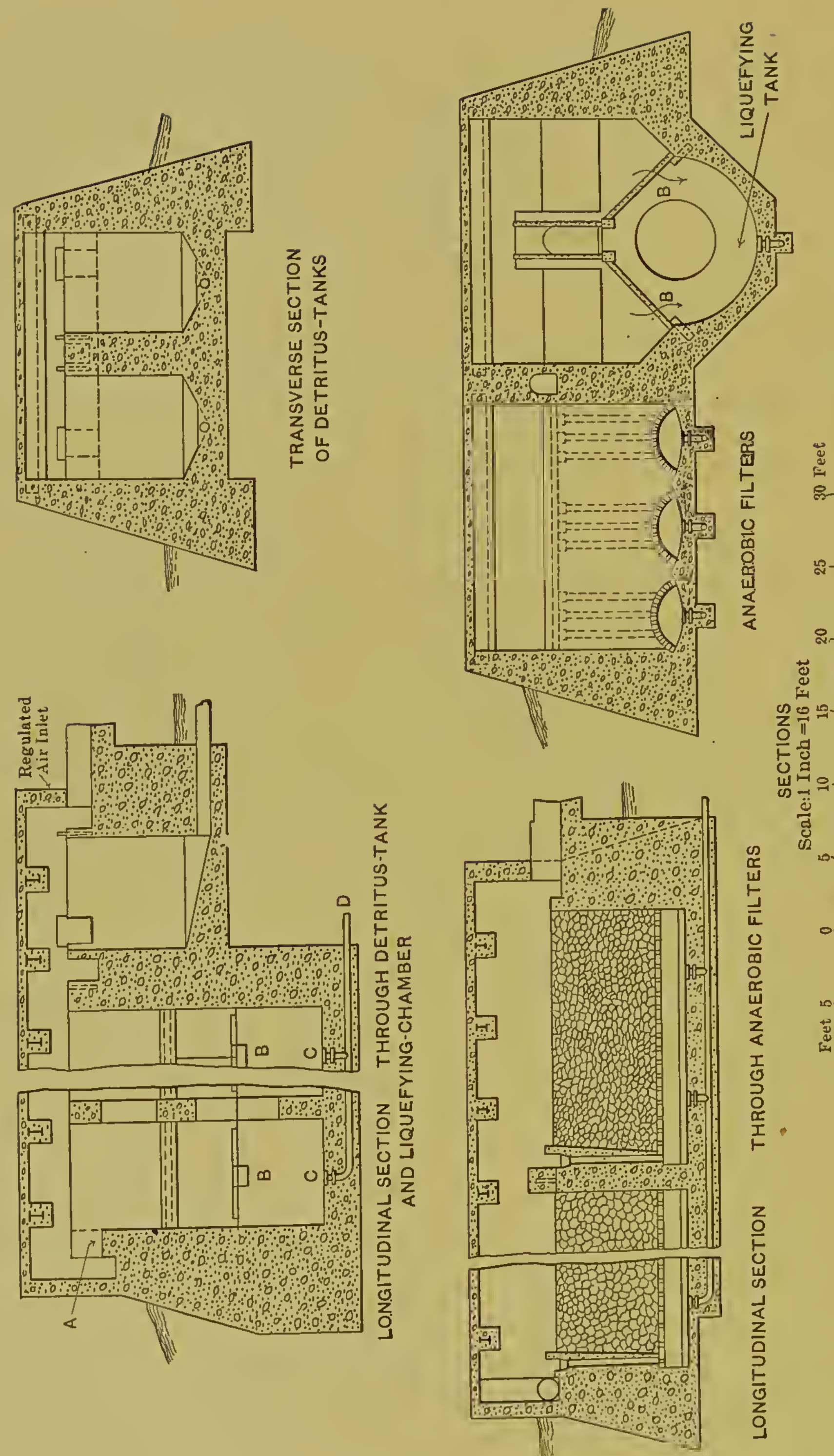


FIG. 47. Sections of the Hampton Hydrolytic Tank and Filters (courtesy of W. Owen Travis).

sludge chamber was to allow sedimentation to go on more freely in the lateral chambers without interference from sludge deposits, from the stirring up due to gas bubbles and from the formation of a thick scum.

Experience with this device led the Hampton workers to focus their attention particularly on the removal of suspended solids. They attempted to carry this further than was possible with the hydrolytic tank proper (which had a capacity of only 5 hours flow) by subsequent treatment of the tank effluent in what was at first an anaerobic filter. Four hydrolyzing chambers were provided, filled with broken flints through which the sewage passed by upward filtration. These beds naturally clogged badly, and Dr. W. Owen Travis, who has been mainly responsible for the Hampton experiments, substituted for the flints a series of plates set parallel to each other at a slight angle with the perpendicular. This change was based on the fact that the removal of suspended solids is not necessarily due to real filtration or straining, but to surface contact or absorption. The evolution of this hydrolyzing chamber from the ordinary anaerobic filter strikingly recalls the development of the Dibdin plate bed from the contact bed. The Hampton tank worked well. The finely divided suspended solids were removed, deposited upon the upper surface of the plates and partially liquefied, so that beneath each plate there was rising a stream of gas bubbles; and the excess of the sludge rolled in small masses off the plates and dropped to the bottom of the tank. The arrangement of the plates in a Hampton hydrolyzing chamber is shown more clearly in the figures of the Norwich plant below.

A somewhat similar device has been studied by Dervaux, according to Dunbar. It consisted of a series of conical surfaces suspended in the tank, and its general construction is indicated in Fig. 48.

Dr. Travis claimed much more for his hydrolytic tank than the removal of the material originally present as suspended solids in the sewage. He pointed out that a considerable proportion (50 per cent in the case of the Hampton sewage) of the organic

matter ordinarily classed as dissolved because it passes through filter paper, is not in true solution but in the colloidal state. Those who care to go into detailed considerations of the exact condition of this material will find a full discussion of the physical chemistry involved in two recent papers (W. O. Travis, 1908; G. L. Travis, 1908). The practical point is that a certain proportion of the organic matter which passes through filter paper is in an unstable state between solution and suspension and may be brought into the suspended condition by appropriate physical processes. Continued shaking causes these border-line solids to pass into colloidal solution. Standing, and particularly surface contact leads to de-solution, or the deposition of the colloids in gross

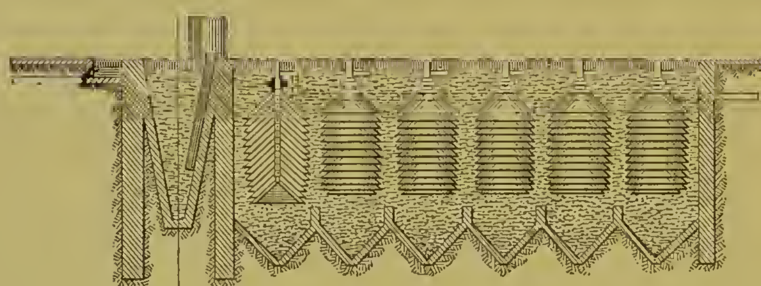


FIG. 48. The Dervaux Tank (copied by permission from Dunbar, 1908).

suspended form. The hydrolyzing chamber with its plate surfaces thus plays an important part in removing dissolved as well as suspended solids.

Upon these observations and experiments Dr. Travis has developed a far-reaching theory of sewage purification which has come to be known as the Hampton Doctrine and which has been defended by him, by Dr. G. L. Travis, by Mr. S. H. Chambers and others in a voluminous series of controversial communications. The Hampton theory of de-solution was first launched by Colonel Jones and Dr. W. O. Travis (Jones and Travis, 1906) four years ago; good statements of the present position of the school have recently been made by G. L. Travis (1908) and W. O. Travis (1908), of which the following paragraph from G. L. Travis is perhaps a fair summing up:

"(1) That the complete operation of sewage purification is a complex one, in which physical, chemical, and biolytical — in its widest sense — factors are severally engaged; (2) that the physical operation manifests itself in removing the suspended, and the more highly complex so-called soluble organic, as well as some inorganic, matters from the sewage; (3) that the chemical operation assists the physical in completing the de-solution of the liquid, and takes its part in the subsequent changes induced by biolysis; (4) that the biolytical operation is only concerned, in a very minor degree, with the purification of the sewage itself, inasmuch as its effects are almost entirely evidenced by the changes occurring in those substances which have been removed from the sewage, rather than by being associated with those which are contained therein; (5) that the biological operations upon the deposited and absorbed solids, whether upon those arising from the suspended, or equally whether upon those resulting from the so-called soluble solids, are always attended by huge, only partially reduced, and practically persistent accumulations."

In essence the claim of the Hampton workers is that the chemical and biological aspects of sewage purification have been greatly over-emphasized. They maintain that the bulk of the organic matter is neither oxidized nor anaerobically reduced in the ordinary processes of sewage treatment, but is merely separated from the liquid by which it is carried. This separation of organic impurities is the main object of sewage treatment; and it can best be attained by simple physical procedures, sedimentation and surface contact, which lead to the deposition of both suspended solids and colloidal organic matter. The attendant or subsequent biological changes are comparatively unimportant.

These views coincide in many respects with those advanced by Bredtschneider (1905), to which further reference will be made in Chapter X. They differ from the commonly accepted views established by Dunbar and his colleagues (Dunbar, 1908) mainly in two respects. In the first place, Dunbar believes that organic matter in solution is removed by adsorption into the surface films, while Travis holds that they are precipitated by mere contact. In the second place, Dunbar maintains that the whole physical process is intimately connected with correspond-

ing chemical and biological changes which lead to the mineralization of the adsorbed organic matter, while Travis considers the latter to be of small importance.

As regards the practical operation of many purification plants Travis is undoubtedly in the right. Intermittent filtration is of all processes the one in which bacterial action seems most complete. Yet at the plant at Brockton, Mass., one of the best intermittent filters in America, eight tons of deposit are removed from the beds for every million gallons of sewage filtered. If the analytical results tabulated in Chapter IX are analyzed it appears that of the nitrogen going onto the beds as albuminoid and free ammonia, 25 per cent appeared in the same form in the effluent and 7 per cent only appeared as mineral nitrogen. These results are obtained by averaging the analyses for the six underdrains and correcting for dilution with ground water by comparison of the chlorine values. They indicate that two-thirds of the organic matter at Brockton is not bacterially purified, but simply strained out and left on the surface of the filter.

Nevertheless the ideal to be aimed at in sewage purification is not merely the separation from the liquid of its organic constituents, but the working over of the latter so that they shall be in a stable and inoffensive form. We know that under favorable circumstances this end can be attained. Scott-Moncrieff's experiments, tabulated in Chapter IX, and the diagram of the Boston experiments on sand filtration (Fig. 65) show that organic nitrogen may be quantitatively converted into mineral form. So with the trickling filter. The Boston experiments discussed in Chapter XI show that the coarse-grain filter used at the Technology experiment station gave out in two years almost exactly the same amount of suspended solids which it received; yet the effluent was generally stable. Of the total organic nitrogen and the nitrogen as free ammonia in the sewage, 75 per cent appeared as organic nitrogen and free ammonia in the effluent and 25 per cent appeared in the mineral form. There was no storage. Retention played only a contributory part in the process. The main factor was the absorption of oxygen and the change of the organic mat-

ter to a stable form. Yet, though Dunbar's theory of the combined effect of physical and biolytic factors seems to fit all the facts satisfactorily, the Hampton workers have undoubtedly rendered an important service in their emphasis on the significance of the physical processes and in their criticism of the over-confidence in biolysis which many experts have unquestionably manifested.

A large plant upon the hydrolytic principle has recently been built at Norwich, England (Collins, 1908), the operation of which should prove of considerable interest.

The general arrangement of the Norwich plant is indicated in Figures 49 and 50. It is designed so that the sewage shall pass first through a detritus tank having a capacity of 53,500 gallons. From the detritus tank it flows to a hydrolytic tank, divided into two lateral sedimentation chambers with a central reduction chamber. The sedimentation chambers communicate with the reduction chamber, below and between them, by narrow ports through which the sludge is to pass. One-fifth of the sewage enters the reduction chamber directly, the amount being regulated by properly adjusted weirs. The capacity of each hydrolytic tank is 260,000 gallons. The effluent from the reduction chamber passes to a hydrolyzing tank with sloping sides and floor having a capacity of 12,000 gallons. Both the sedimentation chambers of the hydrolytic tank and the final hydrolyzing tank are to be equipped with concrete or wooden slabs hung from steel joists, and with their planes parallel to the longitudinal axis of the tanks. In the sedimentation chambers these plates are set vertically, while in the hydrolyzing chamber they are slightly inclined. Four units of this general type are to treat a daily flow of 3,000,000 gallons and the effluent will be treated at first on land and later in filters of special type.

Deep Septic Tanks in West Germany. One of the most important points in the Hampton hydrolytic tank is the separation of the sludge from the flowing sewage and its deposition for further septic treatment in a special chamber provided for the purpose. This principle is carried still further in the design of a septic

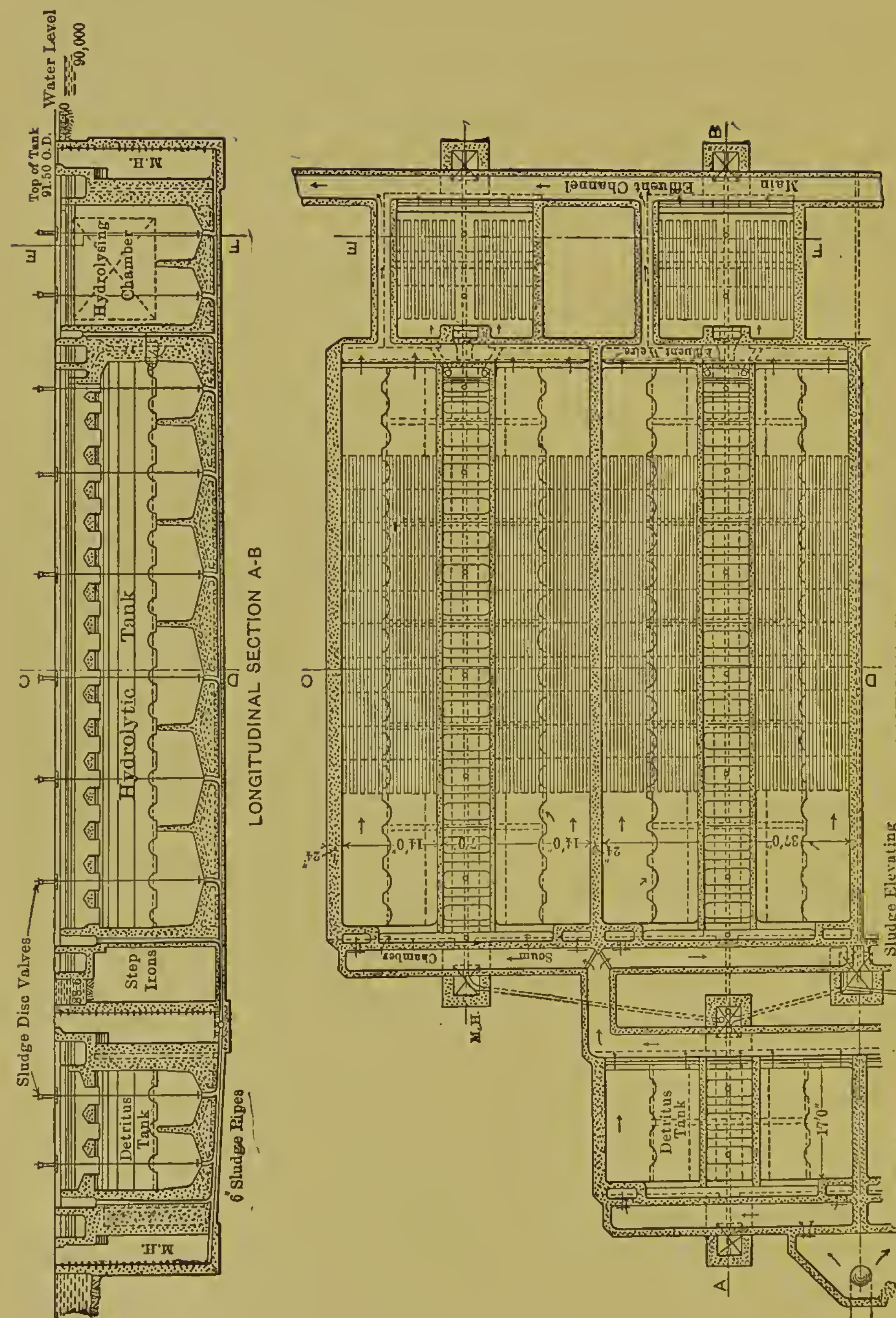


FIG. 49. Plan and Section of Hydrolytic Tank and Hydrolyzing Chamber at Norwich (Collins, 1908).

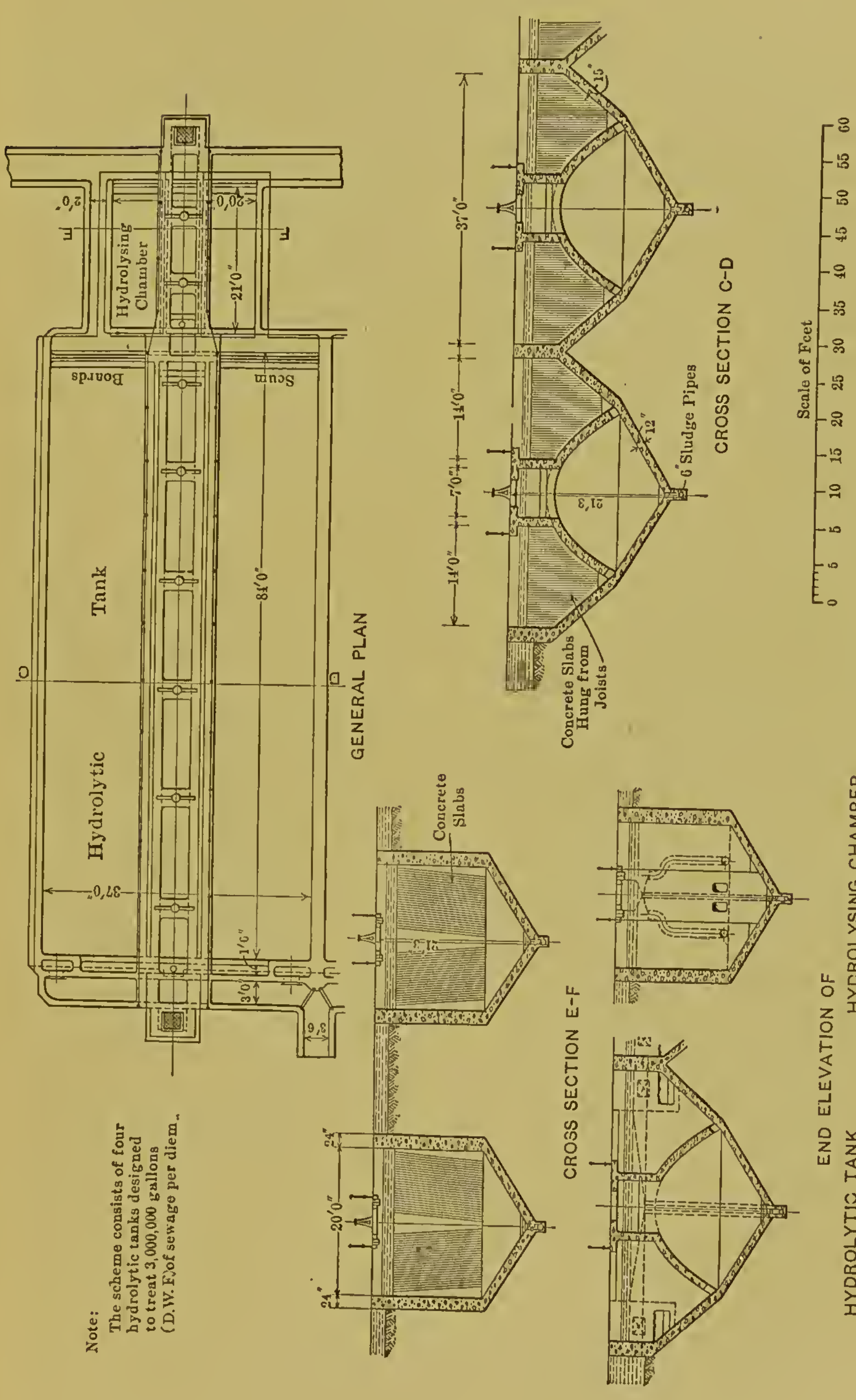


FIG. 50. Details of Hydrolytic Tank and Hydrolyzing Chamber at Norwich (Collins, 1908).

tank which has been strongly advocated by the Emscher drainage board, now commonly known as the Ems or Essen tank. The liquid from Travis's reduction chamber mixes with the effluent from the hydrolyzing chambers. Imhoff in the design of the Essen tank, has tried to keep the flowing liquid and the septic sludge entirely separate (Imhoff, 1909). His tank is of cylindrical form with a conical bottom (see Fig. 51). In the upper portion of the tank is a sedimentation chamber with sloping floors and slots at the bottom to allow the sludge to settle into the liquefying chamber below.

Comparative Efficiency and Costs of Various Preliminary Treatments. A brief review of the respective merits of the different forms of preliminary treatment which have so far been discussed may be useful at this point.

A screen chamber and small detritus tank, for the removal of the larger and heavier foreign bodies which find their way into any sewerage system, may be considered as essential in all disposal plants, except the very smallest. Beyond this, however, it may be an open question whether any other preliminary treatment is desirable, and if so, which of the three chief methods shall be selected.

Intermittent filter beds can certainly be operated successfully with crude sewage. In the eastern part of the United States, where ample areas of sand of the right quality are available for intermittent filtration, preliminary treatment is rarely held to be necessary. In most cases the solids are discharged directly on the surface, where they are partly oxidized and partly accumulate as sludge. In the Middle West, on the other hand, where sand areas are limited and rates of filtration must be high, the use of the septic tank is very general. Barbour, Alvord and Shields are all strong advocates of the practice, and the plants they have installed work very satisfactorily at higher rates than those in use with raw sewage in Massachusetts. It is not at all certain that it would not be cheaper, even when sand is plenty, to remove a considerable proportion of suspended solids by tank treatment instead of scraping them off the surface of the beds.

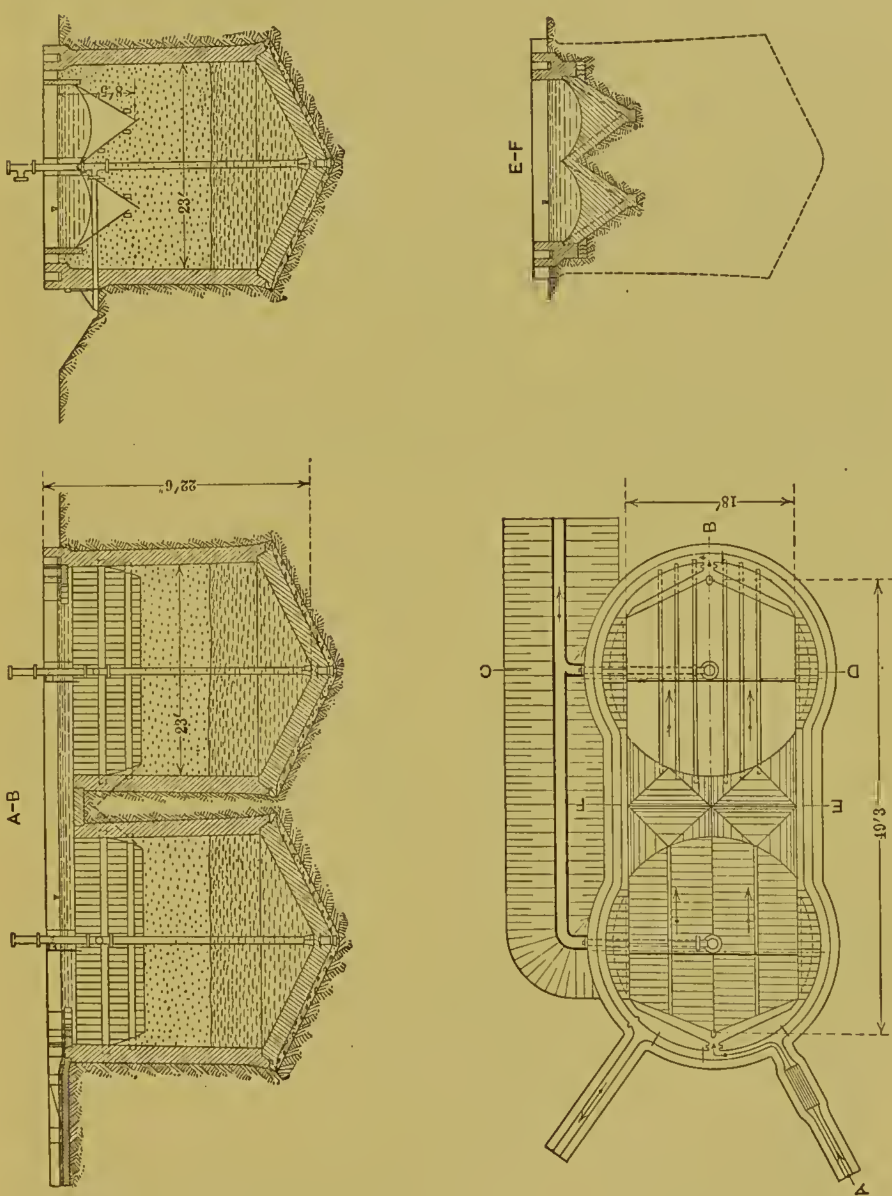


Fig. 51. Plan and Section of the Essen Tank (Imhoff, 1909).

With rapid filters of the contact or trickling type, thorough preliminary removal of suspended solids is almost universal. Even to this rule, however, it is possible that special exceptions may be made when weak sewages are treated on beds of coarse grain, — as suggested by the Boston experiments.

If some form of preliminary treatment be decided on, the engineer has his choice between chemical precipitation, plain sedimentation and septic treatment. Chemical treatment yields the best effluent, but is most costly. Septic tanks cost more than sedimentation tanks, on account of their greater capacity; but they diminish the sludge to be disposed of. The English Royal Commission has made careful cost estimates for the various processes, assuming a sewage of average composition, ample available head, and tanks of simple rectangular design. The principal results of their calculation are tabulated below:

TABLE XXXVII
COMPARATIVE DATA FOR VARIOUS PRELIMINARY TREATMENTS
(R. S. C., 1908.)

Process.	Tank capacity, gallons.	Suspended matter in effluent parts, per million.	Wet sludge, tons per million gallons.	Cost of preliminary process, per million gallons.	Cubic yards filtering material required for final treatment.	Cost of complete process, per million gallons.
Quiescent sedimentation with chemicals	444,440	10–40	15.5	13.83	4,743	20.15
Continuous flow sedimentation with chemicals	833,333	30–60	14.5	12.48	5,533	19.74
Quiescent sedimentation	1,041,660	50–80	10.9	7.93	7,410	16.38
Continuous flow sedimentation	1,041,660	100–150	10.0	6.19	9,540	16.78
Septic tanks	1,200,000	100–150	5.9	6.91	9,540	17.50

All data are in U. S. units.

These estimates are of course very general ones; and the absolute values refer to English conditions only. Nevertheless the table is instructive as a fair illustration of the relative merits of the various processes. It is probably true that septic sludge, allowing for both liquefaction and concentration, will often

amount to a little over 50 per cent of the sludge from sedimentation tanks. Chemical sludge on the other hand is nearly 50 per cent more bulky than sedimentation sludge. The cost of the chemical process taken by itself is probably justly estimated as nearly twice that of plain sedimentation. On the other hand chemical treatment produces an effluent freer from suspended solids and so easier to handle than the effluent from the other processes. Thus the area required for final treatment is much less after chemical precipitation. The cost for the whole process remains somewhat higher however with chemical treatment and is lowest for plain sedimentation.

The various processes are so similar in their general efficiency that local conditions must decide which shall be preferred in a given case. The three determining factors will generally be the character of the sewage, the nature of the final treatment which is contemplated, and the available facilities for handling sludge. For a very small plant serving an institution or a group of dwellings the septic tank is probably to be preferred as lumps of fecal matter in fresh sewage must be broken up and the organic matter carried at least into the first stage of decomposition. For a larger plant, still treating domestic sewage, but older and better mixed, there is greater freedom of choice. The Royal Commission (1908) recommends chemical treatment for strong sewages of this class and septic treatment for weaker ones. In large cities with much manufactural waste, chemical precipitation may be specially indicated, as in the presence of antiseptic substances or large quantities of iron salts.

An even more important consideration is the nature of the final process which is to follow the preliminary treatment. Very elaborate removal of suspended solids would generally be poor economy as a prelude to sand filtration. Again, with coarse-grain stone filters the results of plain sedimentation or septic treatment will generally be good enough to meet the case. Fine filters on the other hand may require more careful preparation; and with such a plant chemical precipitation would often be economically justified.

Finally the available methods of sludge disposal must frequently exert a controlling influence. The English calculations quoted above assume that an acre of land will be used for disposing of 1100 tons of wet sludge; and the value of land is estimated at $484 an acre. Where this and other processes of sludge disposal are attended with special difficulty the septic tank may prove most economical. In general septic sludge is not only less in amount than the sludge from other processes but also less offensive in quality. Furthermore, in the case of small plants at least, tanks operated on this plan need not be cleaned at any special time or with any degree of regularity. It is possible, therefore, to clean the tank at times when the conditions for sludge disposal are most suitable.

CHAPTER VII

DISPOSAL OF SEWAGE SLUDGE

Amount and Character of Sewage Sludge. Sludge, as the word is used in connection with sewage treatment, is generally understood to be the solid matter retained in sedimentation or septic tanks; and the problem of its ultimate disposal presents greater difficulties than any other question connected with sewage treatment. Sludge differs essentially from the deposit which is retained by detritus tanks, and from the solid matter deposited on the surface of intermittent filtration beds, and from screenings. As deposited in the tanks it is a slimy mass, containing 90–95 per cent moisture, has a specific gravity only a little greater than that of water, — 1.04–1.06, — flows by gravity, and cannot be shovelled, or pressed without the addition of mineral matter. It is putrescible and likely to cause an aerial nuisance. The amount formed depends both on the character of the sewage and upon the process used for the preliminary treatment. With the same sewage, chemical precipitation gives the greatest amount of sludge, and the septic tank process the least. With strong English and Continental sewage, using chemical treatment, 25 to 35 tons of sludge, containing 90 per cent of water, may be formed per million gallons of sewage, while with the sewage of American cities 15 to 20 tons would be more nearly an average figure. London sewage yields about 30 tons of sludge, containing 91 per cent of water; Worcester and Providence, U. S. A., 20.6 tons, 93 per cent water, and 16.5 tons, 91.9 per cent water, respectively. The amount of sludge obtained from plain sedimentation is much less, being not over two-thirds of the above amounts, and, since in the septic tank process it may be considered that 20–30 per cent of the suspended matter that is deposited is liquefied or changed into gas, and since also such sludge has gene-

rally a lower water content the quantity required to be handled is about one-third of that produced by chemical precipitation. The fact that so much more sludge is produced by chemical precipitation than by plain sedimentation is not due chiefly to the fact that there is a more complete removal of suspended matter, but rather to the fact that for every pound of chemicals used many times that weight is added to the sludge, one pound of dry lime forming, for instance, not one pound, but ten pounds, in a sludge containing 90 per cent water. The percentage of water that is contained in the sludge makes a much greater difference in the amount that has to be removed from the tanks than any of the other factors. A cubic yard of sludge containing 90 per cent water weighs approximately 1800 lbs., and if diluted so as to contain 95 per cent water, the weight would be increased to 3600 lbs.; 100 tons of sludge, having 90 per cent of water, contains 10 tons of solids and 90 tons of water, and the 10 tons of solids are sufficient to produce 200 tons of sludge containing 95 per cent of water. The reduction in weight of sludge by the withdrawal of water can be easily determined by the following formula:

$$w = \frac{S \times 100}{100 - P}$$

S = original weight of solids in 100 tons of sludge, before drying.
P = per cent of water in sludge whose weight is required (after partial drying).
w = weight of liquid sludge of specified density (after partial drying).

The following table (Raikes, 1908) gives the reduction in weight effected by drying 100 tons of wet sludge containing 95 per cent water:

100 tons of sludge containing 95 per cent of moisture

=	50.00	tons	when	moisture	is	reduced	to	90	per	cent
=	25.00	"	"	"	"	"	"	80	"	"
=	16.33	"	"	"	"	"	"	70	"	"
=	12.50	"	"	"	"	"	"	60	"	"

= 10.00 tons when moisture is reduced to 50 per cent
= 8.33 " " " " " " 40 " "
= 7.14 " " " " " " 30 " "
= 6.33 " " " " " " 20 " "
= 5.55 " " " " " " 10 " "

To determine the volume of sludge in a tank the apparatus used in the Columbus experiments (Johnson, 1905) can be employed. It consists of a glass tube about 2 ft. 6 in. long, and 0.5 in. in diameter, open at both ends, and attached to a wooden rod of sufficient length to reach the bottom of the tank. Through the glass tube there extends a fine wire, at the lower end of which is attached a rubber stopper with the smaller end uppermost, or a rubber ball. The wire is extended up to the top of the rod guided by screw eyes. In making a measurement the rod is slowly lowered through the sewage and sludge to the bottom of the tank. The stopper is then drawn, by means of the wire, into the bottom of the tube, so that when raised to the surface the tube contains a true section of the deposit.

The weight of sludge can be obtained by determining the water in the sample and assuming that a cubic yard containing 90 per cent of water weighs 1800 lbs.

TABLE XXXVIII

ANALYSES OF DRIED SLUDGE FROM CHEMICAL PRECIPITATION, FROM PLAIN SEDIMENTATION, AND FROM THE SEPTIC TANK AT WORCESTER, MASS.

Percentage composition of sludge.	Chemical Precipitation.	Plain sedimentation.	Septic tank.
Volatile solids	47.26	51.04	43.94
Fixed solids	52.74	48.96	56.06
Silica (SiO_2)	25.46	28.59	20.41
Iron sulphide (FeS)	0.02	0.57	16.58
Iron, not as sulphide	5.80	2.45	2.98
Sulphur, not as sulphide	0.44	0.60	0.64
Aluminium oxide (Al_2O_3)	0.57	1.94	7.29
Calcium oxide (CaO)	2.88	0.61	1.14
Magnesium oxide (MgO)	0.74	0.29	0.97
Phosphorus pentoxide (P_2O_5)	0.47	1.71	1.85
Carbon (C)	28.60	31.26	23.95
Hydrogen (H)	4.21	4.46	3.64
Nitrogen (N)	2.77	3.05	3.01

Dried sludge contains from 40 to 60 per cent of organic matter, the amount depending not only on the strength of the sewage, but on the process used for the removal of the suspended matter. At Worcester, the septic tank sludge contained nearly ten per cent less organic matter than the sludge from plain sedimentation, as is shown in the table on page 170. These results are also in accord with data obtained at other places.

Grit Tank Detritus. The solid matter deposited in detritus, or grit, tanks is quite different from sludge. It contains a much greater quantity of mineral matter, the amount of water is usually not much over 35–40 per cent, and it can easily be removed from the tanks by hand or steam shovels. The amount of solid matter deposited in the tanks depends very largely on the rate of flow, and whether or not street washings enter the sewers. Dunbar (1908) states that the total volume of the detritus is generally not more than 1.3 cubic yards daily from a population of 100,000.

At Worcester, with sewage containing more or less street washings, and with an average flow through the grit chamber of 0.5 ft. per second, about 0.12 cubic yards are deposited per million gallons of sewage. The deposit is shoveled out of the chamber, hauled about 600 feet, and covered with earth at a cost of 48 cents per cubic yard. The following data regarding the grit-chamber detritus at Worcester have been furnished by Mr. Almon L. Fales:

TABLE XXXIX

GRIT-CHAMBER STATISTICS, WORCESTER, MASS.

Date of cleaning.	Cubic yards removed.	Million gallons passing.	Cu. yds. per mil. gals.	Cost of removal.	Cost per cu. yd.	Cost per mil. gals.
Dec. 18, 1905...	85	950	.09	$35.73	.420	.038
Jan. 8, 1906...	70	432	.16	34.52	.493	.080
Feb. 26, 1906...	60	506	.12	32.46	.541	.064
Apr. 9, 1906...	70	851	.08	44.53	.636	.052
June 1, 1906...	75	792	.09	43.84	.584	.055
June 27, 1906...	75	603	.12	36.46	.493	.060
July 27, 1906...	70	421	.17	30.85	.440	.073
Aug. 16, 1906...	75	334	.22	32.98	.440	.099
Oct. 5, 1906...	70	589	.12	30.20	.430	.051
Nov. 3, 1906...	80	500	.16	31.15	.390	.062
Totals & averages	732	5,978	.12	$352.72	.483	.059

ANALYSIS OF GRIT-CHAMBER DEPOSIT

Date.	Weight per cubic yard (pounds).	Dry solids (per cent).	Loss on ignition dried sample (per cent).	Organic nitrogen in dried sample (per cent).
Dec. 18, 1905	1,848	65.5	22.0	0.700
Jan. 8, 1906	1,888	64.7	22.5	0.645
Apr. 9, 1906	....	75.4	18.9	0.515
Oct. 5, 1906	2,430	53.3	28.1	0.871
Nov. 3, 1906	1,890	61.8	21.9	0.645
Average	2,014	64.1	22.7	0.675

Screenings. The amount and character of screenings depend largely on the size of screens used. It has been shown in Chapter III that, leaving out of consideration screens which are intended primarily to protect sewage pumps and which have therefore very wide openings, the amount removed from the sewage may vary between 3 and 40 cubic feet per million gallons.

Screenings are usually fairly solid, and contain much more organic matter than grit-chamber detritus. The amount of water in ordinary screenings is not much over 75 per cent, though where very fine screens are used it may amount to 90 per cent. When the moisture does not exceed 75 per cent, screenings can be burned in a destructor, and this method is to be recommended for plants in the neighborhood of a town garbage crematory. If this method cannot be used the screenings should be buried or disposed of with the sludge.

Intermittent Filtration-Bed Deposits. The suspended matter which is deposited on intermittent filtration beds forms into more or less of a mat and can be removed from the surface of the bed by raking or scraping (Fig. 52). In early spring the thickness of this mat may exceed a quarter of an inch, and it can often be removed from the beds in large sheets, as is shown in Fig. 53. This mat consists largely of organic matter, paper, fat and nitrogenous substances. It is easily burned, but unless this is done in a destructor or specially constructed furnace, the odors given off are usually very objectionable. In certain places, as at

Brockton, Mass., contracts have been made for the removal, free of cost, of the total scrapings for use by farmers as a fertilizer. Where this cannot be done, the scrapings may be dug into the ground as in the case of grit-chamber detritus. They are usually only slightly putrefactive, though containing a large amount of organic matter.

FIG. 52. Scraping Intermittent Filtration Beds in Spring, Worcester Sewage Works.

Ultimate Disposal of Sludge. Some authorities have taken the view that sludge should not be formed, but that sewage, after passing through detritus tanks and screens, should be delivered directly on bacterial beds. This is possible, though by no means always advisable, when bacterial purification is brought about on intermittent filtration beds, but when contact beds or percolating filters are used it is the general practice to remove the suspended matter; and consequently sludge-producing tanks form an essential part of most sewage plants.

Notwithstanding this fact, much less study has been given to the subject of the ultimate disposal of sludge than to any other factor connected with sewage treatment, and how tank sludge should be handled has been more or less ignored by most engineers. Many sewage plants have been constructed admirably adapted to produce sludge and non-putrescible effluents, but

FIG. 53. Removal of Mat from Intermittent Filtration Beds, Worcester Sewage Works.

without any adequate means for dealing with the sludge, amounting, according to the method used, to from 5 to 35 tons for every million gallons of sewage.

The methods that are used for the disposal of sludge are, in general: — Disposal at sea, Air Drying, Lagooning, Trenching, Mechanical Drying, Use as a fertilizer, Land Treatment, Burning in destructors.

Sea Disposal. Taking the sludge in steamers and discharging it into deep water is simple and effective, and for large cities situated on the coast or on a tidal river it is the best and cheapest

method. It necessitates, however, specially constructed steamers or barges for carrying the sludge to a point where it can be discharged without danger of polluting shellfish layings, and where the solid matter cannot be carried to any foreshore. This is the method employed at London, Manchester, Salford, Dublin, at two of the plants at Glasgow, and at Providence in the United States.

At London and Manchester, according to Raikes (1908):

"The steamers used for carrying the sludge take about 1000 tons each trip, and are so arranged that when the tanks are empty the bottom is about 6 inches above the light-load line, as also is the top of the sludge when the vessel is fully loaded, and they can thus be emptied by gravity in about 17 minutes, the discharge being regulated by means of large valves controlled from the deck, but as a rule the load is spread over a course taking an hour at a speed of about ten knots.

"The cost of these vessels is from £24,000 to £26,000 each, and in the case of London each trip involves an expense of about £15, which represents about $3\frac{1}{2}$ d. per ton of sludge disposed of, in addition to the cost of precipitation and pumping into the barges."

At Providence the sludge is pressed, and conveyed to barges, which are towed out about ten miles to the United States Government Dump and then discharged. The total cost, including pressing, amounts to $3.43 per million gallons of sewage.

Air Drying. The area required for this method is large, for the depth of sludge should not exceed two to three inches, an amount which, when dried, can be effectually plowed into the soil. The soil most suitable for the purpose is one that is open and porous, to facilitate drainage, and if not underdrained it should be plowed before using. The time required for drying the sludge so that it can be worked into the soil depends greatly upon climatic conditions, for if the area is not underdrained the larger part of the drying is due to evaporation. In hot summer weather the drying may take place in a few weeks, while under other conditions many months are necessary. This method has been employed at Birmingham and at many smaller places. At Birmingham, according to Watson (R. S. C. 1908),

the sludge after a few months assumes the character of rich black soil and can be plowed into the land. The amount of land required is from one to two acres (depending on the character of the soil) per 1000 tons of sludge.

A modification of this method is used in America, especially when the final treatment is intermittent filtration and the sewage has to be pumped. In these cases, to save pumping during the night, the sewage is allowed to collect in a sewage well, and the lower portion of this sewage, when pumping is started the next morning, is much thicker and stronger than the average flow and resembles sludge. This is run, not upon the intermittent filtration beds used for the sewage, but on separate beds constructed on the same principle, except that they contain coarser filling material. The sludge when dried is easily removed from the bed, and when dry is not offensive, and can without much expense be covered with earth. After the removal of the dried sludge the surface of the bed is raked and loosened before receiving another application of the sludge.

Air drying, however, should be restricted to plants where the area to be used is in an isolated situation, on account of the odors given off from the sludge during drying and the possible danger of pathogenic germs being carried by flies and other winged insects, thus endangering the public health.

Lagooning. For the carrying out of this method, sewage lagoons are made by underdraining a given area and surrounding it with earth embankments high enough to contain a depth of from 24 to 48 inches of liquid sludge. The area is carefully underdrained, with drains from 10 to 20 feet apart, and is covered over with some fine material, such as cinder or gravel. When the sludge is run into these lagoons, the water is removed not merely by evaporation, but by draining, a large amount passing through the fine cinder or gravel into the underdrains.

This drainage is apt to be very obnoxious and should be treated on bacterial beds or land before being allowed to run into a watercourse. According to English authorities, the time required for drying the sludge is from two to six months. The volume of the

sludge is reduced by about one-half and the water to about 75 per cent. This allows the sludge to be shoveled and carted away. This method of drying is much cheaper than a mechanical process, as it involves very little expense beyond the cost of renewal of that amount of fine cinder or gravel above the underdrains inevitably taken away with the sludge when the bed is emptied. It cannot, however, as a rule, be recommended for large plants, on account of the time required for drying, during which period an aerial nuisance is likely to be created; and even when the sludge is sufficiently dry to be spaded, it is much more difficult to dispose of than mechanically dried sludge, on account of the larger amount of water it contains.

Trenching. This method consists of running the sludge into V-shaped trenches about 1 foot deep and 2 feet wide at the top, and covering it with earth immediately, or within one or two weeks, by turning back the ridges. The trenches should be cut one or two months before they are liable to be used, so that the earth becomes dry and disintegrated. The amount of land required is much greater than that necessary for lagooning, and two acres of soil of average character is not too much to allow for every 1000 tons of wet sludge, though with sandy soil one acre may suffice. At Birmingham, England, where the amount of wet sludge (94.5 per cent moisture) produced daily is about 1170 tons, three-fourths of a mile of trenches 3 ft. wide at the top and 18 inches deep is often required per day for its disposal (Watson, 1910). The same land, however, it is claimed, can be used again after a period of a year and a half or two years, if in two months or so after covering the sludge with earth the ground is broken up, planted, and, when the crop is removed, again plowed and allowed to remain fallow for about a year.

This method is preferable to either air drying or lagooning; much less nuisance from odors is likely to be created, and there is less danger to public health, since the sludge is covered with earth. It is also a method of absolute disposal, as the sludge does not have to be handled a second time.

It should not be forgotten, however, that this, like the two

preceding methods, is interfered with by rain, snow and frost, and during the winter months in cold climates trenching of sludge is impracticable.

Mechanical Drying. Strictly speaking, drying is not a method of disposing of sludge, but it has the effect of converting the sludge into a form in which it can be sold or given to farmers, or carted and buried, or used for a filling material in isolated localities, or burned in a destructor. Drying is necessary if the sludge is not to be carried out to sea or disposed of by land treatment in the immediate neighborhood of the sewage plant. By mechanical drying the amount of moisture can be easily reduced so that the sludge is sufficiently solid to be readily handled. Though air-dried sludge can be spaded, the drying requires, as has been noted, a large area of land, and is very liable to create a nuisance; so for large works the reduction of moisture must as a rule be brought about by mechanical means. The method commonly used is known as sludge pressing, although recently plants have been built for removing the moisture by centrifugal force.

Sludge Pressing. Sludge pressing reduces the sludge to about one-fifth of its original volume, and the amount of water to 50–60 per cent. The mass thus obtained is much less putrescible than wet sludge, has much less odor, and can be shoveled and easily carted away from the proximity of the works. Pressing avoids to a large extent the nuisances and danger to public health that arise from land treatment of fresh sludge, and renders sludge treatment possible at many places where formerly every foot of available ground had been taken for air drying or sludge lagoons.

By this process the wet sludge is pumped under heavy pressure between sheets of cloth or canvas, which retain the solid matter, but allow the water to be forced through them and out of the chambers of the press. The sludge remains between each pair of cloths in a cake from one to two inches in thickness.

There are numerous forms of sludge presses, although they all consist essentially of a series of cast-iron plates. The plates are

of various shapes, but usually have an area of about one square yard and are mounted on rollers upon side rods which hold the head and tail of the press together. The number of chambers varies from fifty or less to one hundred and twenty-five or more per press. The plates are usually concave on each side and are corrugated. The rims are faced so as to make tight joints when the plates are in contact with each other, thus forming chambers from one to two inches wide, in which the sludge is collected. Each plate has a hole about six inches in diameter in its center. On both faces of each plate are hung cloths which are either sewed together around the central hole in the plate or clamped to the plate at this point. When in use, sludge is admitted under pressure through the central channel and passes out into the bags, the water squeezing through and passing off by the corrugated channels mentioned above.

When no more liquid can be forced out at the pressure used, the plates are separated and the sludge cakes removed from the canvas.

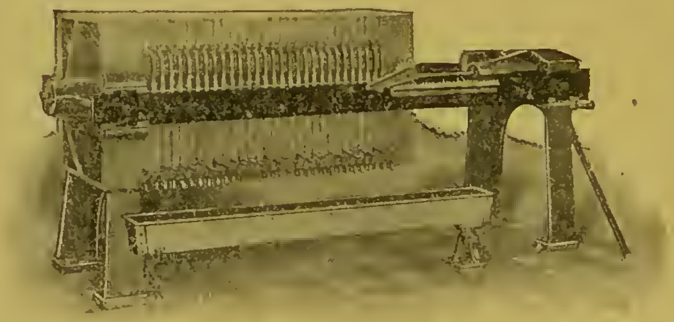


Fig. 54. Sludge Press, for pressing Sewage Sludge (courtesy of John Johnson Co.).

Fig. 54, reproduced by courtesy of the John Johnson Co., shows one form of sludge press used.

The general method used for pressing sludge is to run the sludge from the sedimentation or septic tanks into a sludge well, to add lime in the form of milk of lime, allow the sludge to settle for 12 to 24 hours, draw off the supernatant liquid, and force the sludge by means of compressed air through bar screens into the

sludge presses. The addition of lime is necessary, as the settled sludge, even that obtained from chemical precipitation, is of too slimy a nature to allow of its being pressed into cakes. The amount of lime required varies from 3 to 7 per cent of the weight of the pressed sludge, the least being required for sludge from chemical treatment, the most for septic tank sludge.

The liquid pressed from the sludge contains a large amount of putrescible matter, is strongly alkaline, and gives off a strong odor of ammonia. At plants where chemical precipitation is used it can be run back into the sedimentation tanks, thus reducing to a certain extent the amount of lime required. At other places it can be added to the sewage or treated on fine-grained bacteria beds. The pressure used for pressing the sludge is usually from 60 to 75 lbs. per square inch, and the time required from a quarter to one-half hour. On opening the press, the cake is loosened from the cloth by the use of trowels in the hands of workmen, no mechanical means of doing this having as yet been devised.

The cost of sludge pressing in England (R. S. C. 1908), including interest and sinking fund, is from 50 cents to $1.25 per ton of pressed cake formed; in the United States, from 75 cents to $1.50.

Sludge Drying by Centrifugals. This method of removing water from sewage sludge is of a recent date, the first plant having been erected at Harburg-on-Elbe at the close of the year 1907.

The sludge that is obtained by this process is said often to contain only 50 per cent of water, the amount varying from 50 to 70 per cent; and at Harburg it was not necessary to add lime to the sludge before running it into the centrifugal.

The cost of the drying is given as ranging from 5 to 7 cents per cubic yard of sludge treated. According to "Engineering" (1909): "The machine consists essentially of a drum with a vertical hollow shaft, revolving at a high speed. This drum is fitted with six radial chambers arranged in a star form, each chamber being divided into two sections by a sieve down

the center; one of these sections has parallel sides, and is fitted with valves at the inner and outer ends, these valves being operated by oil under pressure, while the other is of a radial form and contains a drainpipe communicating with an annular canal situated below the drum.

"The method of operation is, briefly, as follows: With the drum revolving, the inner valves open and the outer ones closed, the feed-valve from the tank above is opened and sludge is introduced into the parallel sections of the chambers by way of the hollow shaft and the inner valve-ports. The solid matter present is then immediately carried, by centrifugal force, to the outer part of the chambers, the lighter water being forced back, and thus separated from the solids; this separated water passes off through the sieves into the drainpipes and out of the machine by way of the annular canal. As the process is continuous, the parallel part of the chambers finally becomes filled with a dried mass, the whole of the separated water passing off by the overflow. The inner valves are now closed, and the outer ones opened, when the dried mass is projected outwards against the casing of the machine, striking which, it is broken into small pieces and falls, by way of a funnel, into a receptacle or conveyor below. In the act of being projected outwards, the dried sludge slides past the sieves, and thus frees them from small adhering particles which might tend to close up the meshes and impede the free passage of the water." (Fig. 55.)

At Harburg the fresh sludge, not more than two days old, is run from the sedimentation tanks through $\frac{5}{16}$ inch bar screens and forced into a cistern provided with mechanically operated stirrers and mounted over two of the centrifugal machines. The sludge, in as uniform a condition as possible, is delivered to the different chambers of the centrifugal through a pipe in the center of the revolving chamber. The solid particles of the sludge are thrown against the outer part of the compartments and the water, which is lighter, is held back and runs through the strainers to the outlet channel and back into the sedimentation basin. The space occupied by the water is filled with sludge from the settling

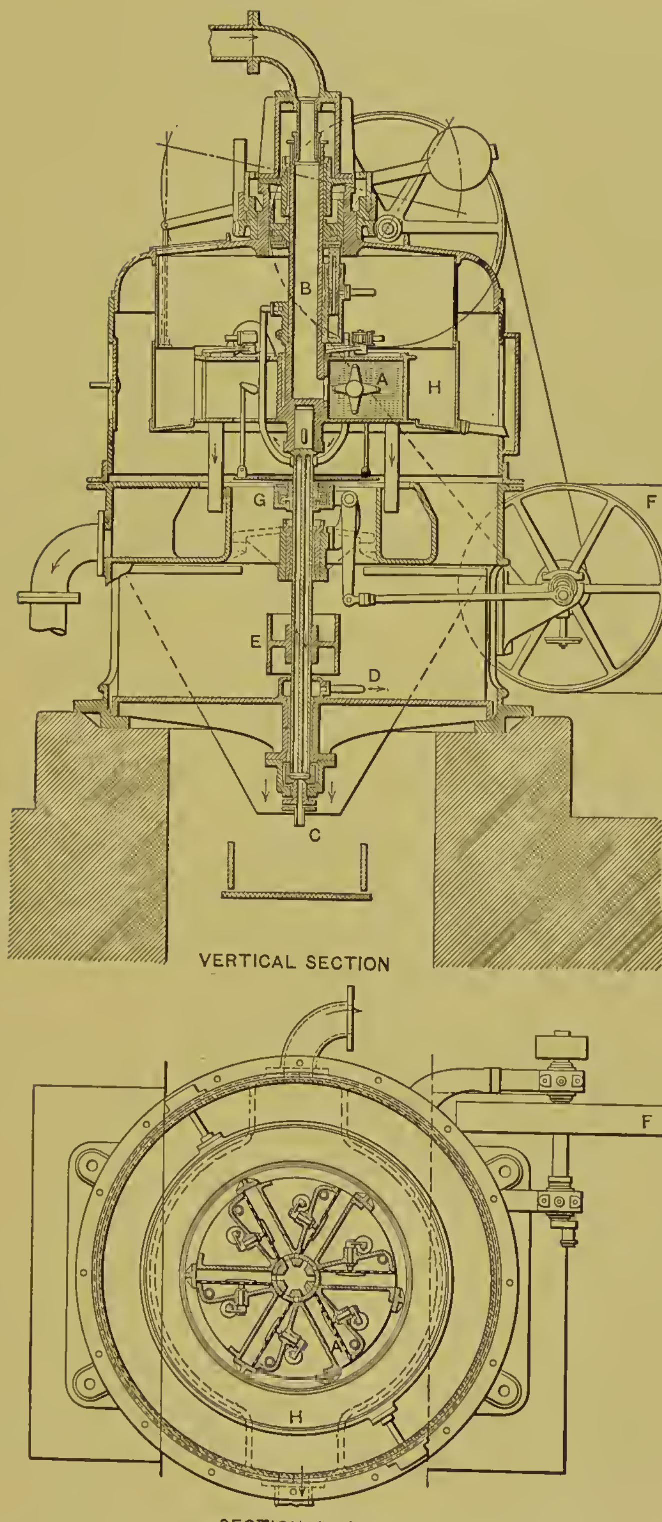


FIG. 55. Centrifugal Dryer for Sewage Sludge (Engineering, 1909).

tank, and in a very short time all the chambers are filled with a dried sludge, which is then thrown out upon the conveyor.

Two of the centrifugals have been continuously in operation at Harburg since 1907, treating from 550 to 700 cubic feet of the sludge per day. The machines are electrically driven, and the energy required to drive the plant is 100 KW per day. At Frankfort-on-the-Main an installation of six machines is in course of erection, to be run night and day and to handle from 260 to 325 cubic yards of sludge in 24 hours. The dried sludge is to be burned in a destructor in connection with the town refuse.

Use as a Fertilizer. This method has always had strong advocates on account of the belief that use should be made of the nitrogen, phosphoric acid and potassium salts contained in the sludge. Sludge undoubtedly has a manurial value, but the amount of manurial constituents is relatively very small, and consequently its value to the farmer as a fertilizer depends to a large extent upon the cost of carriage.

The results obtained by the Royal Commission in their fertilizing experiments, after one year's study, show not only that the manurial constituents of sludge act much more slowly than those of ordinary artificial manures, but also that, unit for unit, the nitrogen and phosphoric acid are of less value than in artificial fertilizers. There is, moreover, the danger of clogging up the soil if too large an amount of sludge is used, especially if the sludge is from chemical precipitation, on account of the large amount of iron and lime salts it contains. Consequently, to find a market for dried sludge is, and always has been, difficult, and a city may consider itself fortunate if the removal of the dried sludge from the neighborhood of the sewage plant can be accomplished free of expense. Two plants, however, in Great Britain seem to have been more or less successful in marketing their sludge. One is at Kingston-on-Thames, and the other the Dalmarnock plant at Glasgow.

At Kingston the chemicals used as precipitants are alumino-ferric, blood, charcoal and clay. The sludge after removal from the tank is pressed, partially dried by heat, and sieved, and after

further air drying during storage is sold under the name of "Native Guano." The analysis of this product, as given in the Fifth Report of the Royal Sewage Commission, is as follows:

TABLE XL

COMPOSITION OF NATIVE GUANO

	Per cent.
Moisture (at about 110° C.)	25.87
Matter volatile on ignition	37.99
Non-volatile matter	36.14
	100.00

ANALYSIS OF NON-VOLATILE MATTER

	Per cent in the sludge.
Grit (*i.e.*, matter insoluble in hydrochloric acid, after ignition)	22.33
Oxides of iron and alumina	10.10
Lime	3.30
Potash (soluble in dilute hydrochloric acid) approx.	0.16
Potash (soluble in water)	0.06
Phosphoric acid (P_2O_5)	1.74
Equivalent to tribasic phosphate of lime	3.30
Yield of nitrogen (total)	1.93
Nitrogen evolved as ammonia on boiling for 2 hours with a dilute solution of potash (0.5 per cent KOH)	0.41

At the Dalmarnock plant at Glasgow the precipitants are lime and ferrous sulphate. The sludge is pressed and dried at a temperature of 165–170° F., passed through a pan mill and marketed under the name of "Globe Fertilizer" at $1.95 to $2.40 per ton in bulk, or $3.40 per ton in bags.

The results of the analyses of this fertilizer, as given by the Royal Commission, are as follows:

TABLE XLI

COMPOSITION OF GLOBE FERTILIZER

	Per cent.
Moisture (at about 110° C.)	22.51
Matter volatile on heating	33.98
Non-volatile matter	43.51
	100.00

ANALYSIS OF NON-VOLATILE MATTER

	Per cent in fertilizer.
Grit, etc. (*i.e.*, matter insoluble in hydrochloric acid, after ignition)	10.75
Oxides of iron and alumina	13.42
Lime	12.09
Potash (soluble in hydrochloric acid) approx.	0.10
Phosphoric acid (P_2O_5)	1.11
Equivalent to tribasic phosphate of lime	2.42
Yield of nitrogen (total)	1.30
Nitrogen evolved on boiling the sludge for 2 hours with dilute (0.5 per cent) potash solution	0.06

A large portion of the pressed sludge is not artificially dried, being sold as pressed cake at 16 to 25 cents per ton in bulk. Though all the sludge from the Dalmarnock outfall is disposed of to farmers, it is to be noted that sludge produced at the Dalmuir and Shieldhall outfalls is carried out to sea.

Land Treatment of Dry Sludge. Dry sludge can be easily disposed of by digging it or plowing it into the land. This can be done as soon as the sludge is delivered from the presses or centrifugals, and the area required is not over one-fifth that necessary for trenching wet sludge, as the volume of the sludge is reduced four-fifths during drying. Probably the area required is much less than the above figure, as the same land can be used more frequently than when wet sludge is applied. Winter weather again does not interfere seriously with the land treatment of dry sludge, as sludge containing 50–60 per cent of moisture is not very quickly putrescible and can be tipped and allowed to remain uncovered for considerable periods of time without creating a nuisance.

When there is not sufficient land available in the immediate vicinity of the plant for digging or plowing in of the sludge, or where the farmers cannot be relied upon to remove it, the sludge must be carried away to some isolated locality or burned. Dry sludge can also be used as a filling material if the locality selected is sufficiently distant from traveled roads and dwellings, and there is no probability of the land being used for building purposes. Where this method is possible, it is, next to sea disposal, the most satisfactory way of treating sludge.

At Worcester, Mass., the sludge, when loosened from the cloths of the sludge presses, drops into conveyors, running under and parallel to the presses. The conveyors consist of iron troughs in which 6-inch iron lugs attached to endless wire cables travel in one direction and drag the sludge through the troughs and drop it into cars at the outer end. The sludge cars, shown in Fig. 56, are hauled by an electric motor car to a deep isolated valley three-quarters of a mile from the press house, and the sludge is dumped from a trestle, the location of this trestle being

changed as the occasion demands. Such favorable conditions as these for the disposal of the dried sludge are somewhat unusual. (Fig. 57.)

Burning of Sludge. Many experiments on a large scale have been made in regard to the burning of sludge, and it has been shown that there is no inherent difficulty in burning sludge when the moisture content is not over 60 per cent, and that sludge

Fig. 56. Electric-motor Sludge Cars, Worcester Sewage Works.

containing 75 per cent of water can be burned with careful manipulation.

At Worcester, Mass. (Eng. News, 1892 *b*), in 1891 several experiments were made with the burning of sludge in a furnace with low chimney, both with and without forced draught. In one of these experiments, 45 tons of sludge, containing about 46 per cent moisture, were burned, three cords of wood being used as fuel. The clinker formed was a semifused mass, due to the

large amount of iron contained, and was not difficult to remove from the furnace. The cost, chiefly for manual labor in collecting the sludge from the sludge beds and bringing it to the furnace, amounted to $3.00 per ton of dry sludge. In another experiment sludge containing about 72 per cent moisture was burned at the rate of 2.2 tons in 9 hours, aided only by a very small amount of fuel.

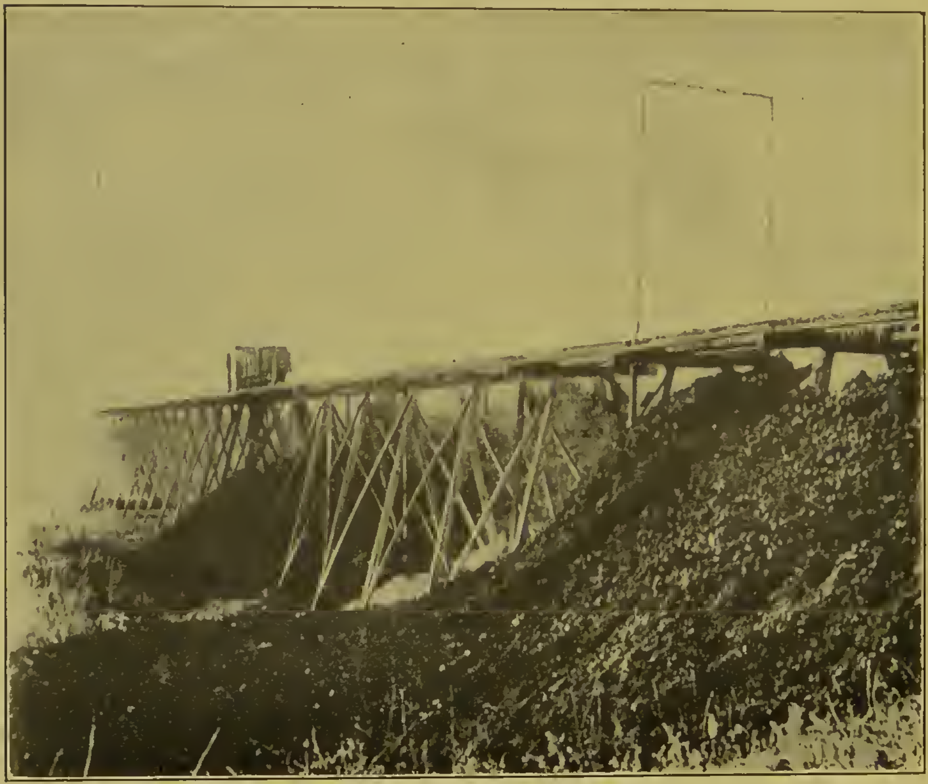


FIG. 57. Dumping Pressed Sludge, Worcester Sewage Works.

At Coney Island (Eng. News, 1892 *a*), a large pleasure resort near New York, burning sludge was also tried about 1890. The sludge, formed by treating the sewage with ferric chloride and lime, was mixed with sawdust and burned in a destructor. The sawdust was to take up the moisture in the sludge and to render it more easily handled. The process was only continued, however, for a few years.

In England, at Ealing-on-Thames, sludge has been successfully disposed of by burning. Ealing is about 9 miles from London, and has a population of about 35,000. The sewage is treated with lime, clay and aluminium sulphate. In 1883, by the advice and under the supervision of Mr. Charles Jones, a destructor for the burning of the town refuse and sludge was built. The destructor plant consisted of seven cells of Fryer's patent destructors with a grate area of 5 by 5 feet each, to which, in 1889, was added a fume crematory, an invention of Mr. Jones's, which consisted essentially of a chamber in which a fresh coke fire was maintained, over which all the gases arising from the fires in the destructors were passed before allowing them to escape into the air.

The treatment of the sewage sludge to prepare it for burning was as follows (Fuertes, 1897, and Eng. Rec., 1900):

There were three sheds, about 30 by 100 feet each, with concrete floors and roofs supported on posts, about 10 feet apart, around their peripheries. The sheds were entirely open, except for movable sides formed of boards laid horizontally around the sheds inside of the posts. The floors were drained by pipes to a sump-well. The refuse wagons drove into the sheds and dumped their contents on the floor. The attendants then arranged the refuse along the sides and ends so as to form a big bed or pond, into which, when the sides were raised about 4 or 5 feet high, they pumped about 2 feet of sewage sludge. Upon this sludge was spread 6 inches in depth of refuse, upon which was pumped about 2 feet more of sludge, and this was covered with another 6 inches of refuse. The mixture was then allowed to stand until the free water drained out and ran into the sump-well. From this it was pumped to the sewage settling tanks to be re-treated.

After the mixed sludge and refuse had drained a sufficient length of time it was taken in small iron dump cars to a hydraulic elevator which raised it to the top of the destructors, where it was dumped. There it was fed to the fires with about double its volume of ordinary refuse. One-third of the daily collection

of refuse was apportioned to be mixed with the sludge, the remaining two-thirds being fed with the mixture as described above. The clinker taken from the fires was used principally in road-making, but the finer portions which fell through the fire bars were used for concrete manufacture.

In 1902 this method was given up and the sludge was pressed before burning. The pressed cake, containing about 60 per cent of water, was conveyed by a hydraulic lift to the destructor furnaces and then mixed with 1.5 to 2 parts dry house refuse to 1 part sludge cake. The steam generated was utilized for various purposes at the plant. From 2500 to 3000 tons (R. S. C., 1908) of pressed sludge are thus disposed of per annum.

Mr. Charles Jones states that the cost of burning generally works out at from 36 to 40 cents per long ton, but he believes that there is no real cost, but rather the reverse, as the clinker that is obtained is used for various purposes, as wallings, concrete, bacteria beds, etc.

Extensive experiments have also been carried on by Campbell at Huddersfield (R. S. C., 1908). The combustion plant there consists of a two-cell Horsfall destructor, each cell having a grate area of 30 square feet with a drying hearth at the back. Each cell is fitted with a steam blower. Mr. Campbell's original intention was to mix the pressed sludge with an equal amount of town refuse, but on account of the difficulty in delivering the town refuse at the sewage works, coke breeze was used, and for part of the time gas-works clinker, containing a fair amount of coke and combustible matter, was substituted. The pressed sludge is tipped into feeding bins at the back of the furnace and there mixed with the coke breeze or gas-works clinker in the proportion of one part fuel to five parts sludge cake. The fumes given off from the pressed sludge in drying pass over the hottest part of the fire in the destructors before they escape to the chimney. The average cost of burning the pressed sludge (R. S. C., 1908) was 63 cents per long ton.

Although, as these experiments show, the burning of sludge

must still be considered in the experimental state, there is no question that sludge can be thus disposed of without offense and at a cost which is not prohibitive.

In America, where public attention is being rapidly aroused to all questions pertaining to public health, including the disposal of sewage and the proper removal and destruction of garbage, a careful study of crematories, where both dry sludge and garbage can be consumed, would probably prove of advantage to the sanitary engineer.

Destructive Distillation of Sludge Cake. Comparatively little attention has been paid as yet to this subject. Experiments, however, have been made in America and England regarding the products formed when sludge is heated in retorts, and they have shown that a large part of the nitrogen of the sludge is recovered in the form of ammonia, and that, as with the destructive distillation of organic matter in general, the other products are a more or less luminous gas, tar, oil, and a coke-like residue.

In England, Mr. George Watson (R. S. C., 1908) reports the results obtained from an experimental plant. The average amount of ammonium sulphate obtained in six different experiments from one long ton of absolutely dry sludge was 105.9 lbs., maximum 124 lbs., minimum 81.7 lbs.; the amount of residue left in the retorts averaged 47 per cent, maximum 50, minimum 42 per cent. The amount of oil was very variable, depending on the heating of the retort, the maximum amount obtained being 30 gallons. The gas was not measured, but was used for heating the retorts.

In America, the Massachusetts State Board of Health (1910) have made experiments in destructive distillation, using sludge from chemical precipitation tanks, plain sedimentation tanks, septic tanks, and settling tanks receiving the effluent of trickling filters. The distillation process was carried out on two or more samples from each source, four hundred grams of dried sludge being used in each case.

The results obtained are tabulated as follows:

TABLE XLII

ANALYSES OF SLUDGES USED FOR DESTRUCTIVE DISTILLATION, PER CENT OF COKE FORMED AND AMOUNT OF NITROGEN IN COKE

(Mass., 1910.)

	Composition of sample before distillation. (Per cent.)			Per cent of coke produced.	Per cent of Nitrogen.*		Per cent available P_2O_5 in coke.
	Total nitrogen.	Loss on ignition.	Fats.		Found in coke.	As NH_3 in washer	
Lawrence sludge †	3.36	36.8	12.8	63.5	.11	.586	1.33
Andover sludge †	2.14	46.6	27.5	59.5	.67	.226	1.33
Clinton sludge †	2.36	74.4	7.7	44.5	.72	.404	1.44
Brockton sludge †	1.76	46.0	6.2	60.5	.94	.137	1.17
Worcester sludge ‡	1.19	44.5	3.2	54.0	.09	.544	1.67
Septic tank sludge	2.46	47.9	8.3	68.5	.27	.497	1.15
Trickling filter sludge	2.10	48.3	4.9	62.0	.56	.809	1.31
Sludge from evaporation of sulphite pulp liquor	....	87.3	....	32.0	....	....	....
Peat	2.54	92.0	....	49.0	.70	.700	0.31
Sawdust	0.00	....	....	25.0	....	.000	....
Wood pulp	0.00	....	....	25.0	....	.000	....
Soft coal §	....	96.8	....	77.3	....	.222	....

* Per cent by weight of total sludge taken.
† Settled sewage sludge.
‡ Chemically precipitated sludge.
§ Average of four kinds of steam and gas coal.

RELATIVE VOLUME AND COMPOSITION OF GASES PRODUCED BY DESTRUCTIVE DISTILLATION OF SEWAGE SLUDGE

	Cubic feet of gas per ton of sample.	Per cent.						
		CO_2	Illuminants.	O	CO	H	CH_4	N
Lawrence sludge *	4,900	4.4	2.2	0.3	30.7	34.9	18.6	9.1
Andover sludge *	6,400	7.4	15.1	0.6	14.3	22.9	34.3	5.4
Clinton sludge *	9,100	8.3	6.7	0.0	20.4	33.2	24.5	7.0
Brockton sludge *	6,000	16.5	21.4	0.2	10.3	22.6	29.1	0.2
Worcester sludge †	8,100	14.2	4.9	0.3	29.8	32.6	16.2	2.2
Septic tank sludge	4,900	7.5	1.0	0.1	24.3	44.0	13.0	10.2
Trickling filter sludge	6,000	20.2	17.4	0.3	6.6	32.7	22.8	0.0
Sludge from evaporation of sulphite pulp liquor	11,000	21.6	2.1	0.0	20.0	42.0	12.0	0.3
Peat	8,400	39.0	4.7	0.2	11.0	28.0	17.1	0.0
Sawdust	12,700	18.4	4.8	0.3	28.2	26.9	19.1	2.3
Wood pulp	12,000	23.5	1.4	0.0	16.4	44.5	13.3	0.9
Soap grease	5,400	6.8	44.5	0.0	6.2	15.8	26.7	0.0
Soft coal ‡	10,200	1.6	2.0	0.1	5.2	62.3	25.7	3.2
Lawrence illuminating gas §	....	3.4	9.1	0.0	21.5	42.5	19.7	3.8

* Settled sewage sludge.
† Chemically precipitated sludge.
‡ Average of four kinds of steam and coal.
§ From gas company pipes at experiment station.

Cost of Sludge Disposal. Very few data are obtainable as to the cost of sludge disposal in America, by the various methods described. A rough comparison as to the relative cost of sludge disposal in England can, however, be obtained from the following table (R. S. C., 1908):

TABLE XLIII

COMPARATIVE COST OF THE VARIOUS METHODS OF SLUDGE DISPOSAL, AS INSTANCED BY THE EXAMPLES GIVEN (R. S. C., 1908.)

Method of sludge disposal.	Number of examples.	Maximum and minimum cost of the process, in cents, per ton of sludge containing 90 per cent of water, including interest and sinking fund and all other charges.		Average cost of process, in cents per ton of sludge containing 90 per cent of water, including interest and sinking fund and all other charges.
Covering land with sludge.	3	Minimum	2.64 cts.	4 cts.
		Maximum	5.6 cts.	
Sea disposal	6	Minimum	8.2 cts.	10 cts.
		Maximum	13.8 cts.	
Trenching in soil	3	Minimum	8.0 cts.	10 cts.
		Maximum	14.0 cts.	
* Pressing, Group I	10	Minimum	9.6 cts.	12 cts.
		Maximum	14.6 cts.	
† Pressing, Group II	11	Minimum	15.4 cts.	23 cts.
		Maximum	25.2 cts.‡	
Pressing and burning	1	(excluding interest and sinking fund)	26.6 cts.	36 cts.§

* In this group are included towns having a population of about 30,000 and upwards, where the preliminary treatment consists either in chemical precipitation followed by sedimentation or in simple sedimentation, and where the sewage does not contain manufacturing waste of a kind likely to necessitate the addition of an unusual quantity of lime to the sludge before pressing.

† In this group are included towns having a population under 30,000 and also places where 5 to 20 per cent of lime (calculated on the pressed cake) has to be added to the sludge before pressing, either because the sewage contains much grease, or because septic tank sludge has to be dealt with.

‡ Owing to the great variations in the actual figures for interest and sinking fund, mainly because of the different dates at which the sewage works were constructed, we have been obliged, in dealing with "Pressing," to take an approximate average figure for this item, viz., 18 cents per ton of pressed cake containing 55 per cent of water, or 4 cents per ton of sludge containing 90 per cent of water.

§ This figure is an estimate.

Though, as has been stated, the method which should be chosen for the disposal of sludge depends on local conditions, as,

for instance, the size of the plant and its location, yet the following general statements can be made:

For small plants not situated in the near vicinity of dwellings, land drying or lagooning is, as a rule, advisable, the dried sludge being given to farmers or disposed of by digging it into the soil. For medium-sized plants, trenching is a much better method, as it avoids the creation of a nuisance and the re-handling of the sludge. For large plants, if situated near the coast, sea disposal will usually be found the best and least costly method. For large inland plants mechanical drying must be resorted to, with subsequent disposal by gift to farmers, by digging into the soil, or by filling up of low land in very isolated localities, and where these methods are not available, by burning it, mixed with house refuse or a small amount of fuel, in a destructor.

It should, however, be mentioned that where the sludge contains certain kinds of trade waste, as, for instance, wool grease, as at Bradford, England, special methods of treatment may often be advisable.

CHAPTER VIII

PURIFICATION OF SEWAGE BY BROAD IRRIGATION AND SEWAGE FARMING

The Beginnings of Broad Irrigation. Irrigation is the most natural method of sewage disposal where large rivers or lakes are not at hand. The living earth absorbs the liquid and digests the solid constituents of sewage. The annual disappearance of manure in fertilized land shows how complete this process may be; and it illustrates the fact that the organic matter thus digested is not only rendered harmless, but is changed into a form in which it serves as food material for the higher plants.

The distinguished English sanitary engineer, Baldwin Latham, believed that he had discovered evidence of sewers and irrigation areas in the ancient city of Jerusalem. It is certain that the Chinese and Japanese have disposed of sewage by this method for thousands of years. The excreta in their crowded countries are either collected in dry closets and spread directly on the fields, or they are discharged into small streams or canals, which are later used for irrigation.

In Europe what was practically sewage irrigation was practiced in a few localities at a very early date. Lausanne, for example, discharged all its sewage into a small brook, which was equivalent to a main collecting sewer. The use of the water of this brook for irrigation was regulated by law in 1400 (Ronna, 1872). In the fifteenth century, too, Milan and other cities in Lombardy constructed canals for carrying their waste waters out to farm lands in the neighborhood. The first irrigation area in Europe designed to take house sewage directly appears, however, to have been at the town of Bunzlau in Prussia. Here, in 1559, a water supply was brought in wooden pipes from a famous spring near by. Sewers were installed, with individual house connections,

and the sewage was carried to an irrigation area of about 35 acres in extent. The land was privately owned, and special ordinances were drawn up regulating the hours when each farm should receive its supply of sewage (Du Marès, 1885). For over three hundred years this irrigation area was in use without any sign of deterioration.

Another famous installation is the farm at the Craigentinny Meadows, which receives the sewage of the city of Edinburgh. Irrigation of some sort appears to have been practiced here from time immemorial; but in 1760 an extensive system of sewers was constructed which discharged on open meadows near the sea. The land was originally a waste of sand dunes, but the 250 acres irrigated with sewage have yielded heavy crops of hay and Italian rye grass for a hundred and fifty years.

Sewage Farming in England. The real development of broad irrigation came, however, in England; and it dated from the wave of sanitary reform which swept over that country in the middle of the last century. Mr. A. D. Adrian, in testifying before the Royal Commission on Sewage Disposal (R. S. C., 1902), divided the history of the subject into three periods, from 1842 to 1857, from 1858 to 1870, and from 1870 to the present day. The dominant idea of the first period was the necessity for removing excretal matters from the vicinity of dwellings without much reference to their ultimate fate. This was the period of sewerage. The report of the Health of Towns Commission in 1844 sounded the first bugle call of the great campaign for public health. It revealed an astonishing accumulation of filth of all sorts in the cities of Great Britain, and the administrative ability of England was actively devoted to remedying the conditions revealed. The construction of sewer systems and the destruction of privy vaults and cesspools made rapid progress. Sewerage systems inevitably, however, led to the next step, — sewage disposal; and the consideration of this question was the characteristic of the next decade.

In 1857 a new Royal Commission was appointed to inquire into the best means of disposing of sewage (Sewage of Towns

Commission). In its first report, in 1858, this commission discussed sewage irrigation with some fullness. The supposed dangers from "the creation of largely extended evaporating surfaces from sewer water," which had retarded the development of irrigation, were minimized. The system in use at Milan, where the liquid refuse of the city was conducted by a canal called the "Vettabbia" to an irrigation area of about 4000 acres, was described as a proper model. In its final report, in 1865, this commission concluded that "the right way to dispose of town sewage is to apply it continuously to land, and it is only by such application that the pollution of rivers can be avoided." According to Mr. Adrian, the dominant note of this period from 1858 to 1870 was the growth of the idea that irrigation, instead of being a menace to health, was the ideal system of sewage disposal.

Mr. Adrian's third period, initiated by the first report of the Rivers Pollution Commission, in 1870, was distinguished mainly by a difference in the stress laid upon the aim of sewage treatment. In the earlier period the main object was to dispose of the sewage without local nuisance; after 1870 the protection of the purity of streams took on a more and more predominant part in the discussion. Through both periods, however, from 1857 until ten years ago, broad irrigation was the general method of sewage disposal in England. Even up to two years ago (1908), when in its Fifth Report the Royal Commission on Sewage Disposal (appointed in 1898) at last concluded that sewage could be purified by "artificial filters" as well as by irrigation, the Local Government Board of Great Britain held that land treatment was the only proper method of purification. With the desire to dispose of polluting material, there grew up in the early days a parallel interest in sewage farming which was sometimes almost as important as the sanitary one. Liebig and his followers laid great stress upon the danger of an exhaustion of the world's nitrogen supply, and irrigation was hailed as a profitable method of turning organic wastes into valuable crops. The two conceptions are well combined in the definition of irrigation

by the British Metropolitan Sewage Commission of 1884 as "the distribution of sewage over a large surface of ordinary agricultural land, having in view a maximum growth of vegetation (consistent with due purification) for the amount of sewage applied."

The construction of extensive sewage farms began at once in England after the report of the Sewage of Towns Commission in 1858, and progress continued at a rapid rate during the next two decades. Ronna, in 1872, described sewage farms at Aberdeen, Rugby, Watford, Carlisle, Penrith, Banbury, Warwick, Worthing, Bedford, Croydon and Norwood, Tunbridge-Wells, Redhill and Reigate, Birmingham, Lodge-Farm, Parsloes, Aldershot and Romford. Du Marès estimated in 1883 that over two hundred irrigation areas of various dimensions were in operation in different parts of England. Many of the sewage farms now in use were laid out in these early days. The farm at Croydon, for example, dates from 1861, and that at the military camp of Aldershot from 1864.

The English enthusiasm for broad irrigation was not slow in spreading to the continent. The first sewage farm operated there on a large scale was at Dantzic. In 1869 a contract was signed with an English engineer, Alexander Aird, by which the sewage of the city and 1300 acres of land were ceded to him for a term of thirty-two years, the entire maintenance of the sewerage system being in his charge. The operation of this plant had a special interest on account of the severe winter weather to which it was subjected; and its complete success from both sanitary and economic standpoints had great influence in establishing the practice of broad irrigation in Germany. Experiments on sewage farming were begun at Paris by Mille and Durand-Claye in 1865, and important studies of the chemical and bacteriological principles involved were later carried out by Schloesing and Müntz and others. Sewage irrigation at Berlin does not date quite so far back. Operations were first begun at Osdorf in 1876, after a long investigation under the leadership of Rudolf Virchow. An area at Falkenberg was added in 1879, and two areas at Gross-

beeren and Malchow in 1882; and the farms are to-day the largest in the world.

Construction and Operation of Irrigation Areas. The purification of sewage on land is carried on in England by a combination of two processes, — broad irrigation and land filtration. The earlier farms were so designed that the sewage was allowed to run continuously over the surface of the soil in as thin a film as possible, without being specially encouraged to pass downward. As a rule, the sewage was brought to the highest level on the farm, and thence distributed by open carriers, following the contours; these, in turn, discharged through lateral carriers or ditches.

On many farms the plots for receiving sewage were arranged on several different levels, so that after flowing from the surface of one it could pass over a second and perhaps a third. This is broad irrigation, pure and simple. On most of the farms, however, special areas, sometimes including the greater portion of the land, have been leveled and underdrained, so that downward filtration plays a predominant part. Under these conditions the net rate is higher than with surface irrigation alone; but it is even then rarely much over 10,000 gallons per acre per day. Land filtration in England is, therefore, not the same process which we call intermittent filtration in the United States. Here, we limit the latter term to a plant which purifies sewage at fairly high rates (30,000–100,000 gallons per acre per day), generally without cropping; although certain plants like that at Framingham, which are regularly cultivated, are somewhat intermediate between the American and English types. In England a rate of 30,000 gallons per acre is an exceptional maximum, and the growing of crops is always an important part of the process.

Aside from the laying out of the land the details of the sewage farm are few and simple. As a rule, the sewage is subjected to sedimentation and screening as a preliminary treatment. The Royal Commission in its last report (R. S. C., 1908) points out that "Porous sandy soils, worked as filtration farms, may be able to treat crude unsettled domestic sewage without detriment, but, even in those cases, there is the possibility of nuisance arising

from the decomposition of sewage solids on the surface of the soil, and such solids may cause damage to crops."

On the farm itself, the surface must be made fairly level, in order that sewage may not collect in pockets; and when land filtration is used the surface must be carefully evened. The sewage is distributed by open carriers of stone, brick, concrete or half-pipe, the finer ramifications of the system being usually simple trenches. The importance of proper distribution is well indicated by the history of the sewage farm at Aldershot. From 1880 to 1895 this plant was badly managed and became a nuisance. In the latter year Colonel Jones took charge, and, largely by re-grading and by care in laying our distributing channels, brought the farm into such excellent condition that the managers of a military hospital close by are quite satisfied to have it almost beneath their windows.

It is common, as pointed out above, to prepare favorable sandy areas on the farm for treating a portion of the sewage intermittently at a somewhat higher rate. Provision should also be made for storm water, either by constructing special roughing filters or by allowing an extra area of land. Barwise (1904) gives the diagram reproduced in Fig. 58 as an ideal plan for a sewage farm. The sewage passes from the detritus tanks at *A* to one of the seven intermittent filters or to one of the twenty-eight irrigation plots. Roughing filters at *C* take the first excess of the storm water flow and the osier beds along the bank of the river receive the remainder. If for any reason ordinary concentrated sewage cannot be treated on the beds, it is purified by artificial filters of crushed stone at *B*, which should have a capacity equal to one day's average flow.

The surfaces of the filtration areas are generally laid out in ridges and furrows, to further facilitate the absorption of the sewage. The distribution from the smaller trenches is controlled by hand, a stop board mounted on a semicircular piece of sheet iron and provided with a handle, being pressed into the soil to dam the sewage stream wherever desired.

It is generally necessary to keep down the ground-water level

by digging open ditches at frequent intervals or by laying a system of underdrains. Underdrains are of course to be preferred, except in clay soils, where they may do more harm than good by promoting the formation of cracks and fissures. In England they are laid three or four feet below the surface, and at considerable distances apart, generally from thirty to fifty feet or more between centers. Raikes (1908) gives 660 to 1320 12-inch lengths of pipe as the standard allowance per acre. The

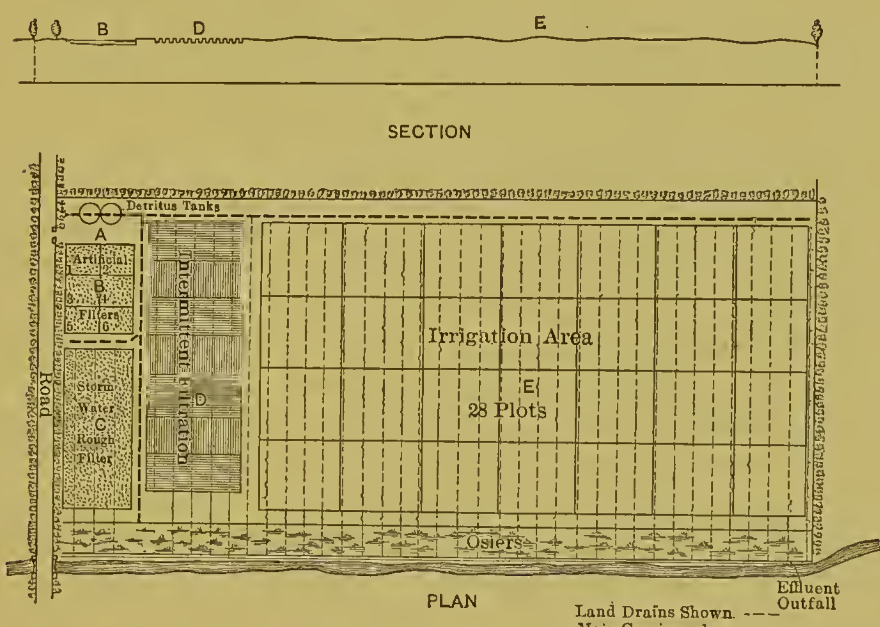


FIG. 58. Diagram of a Sewage Farm (copied by permission from Barwise, 1904).

gradient of the pipes and their size must naturally be adjusted to local conditions of fall, etc., but 4-inch pipes are the convenient size for the smallest of them. The pipes should be surrounded by fine ashes, gravel or surface soil in order to prevent leakage from above.

The commonest crop on the English sewage farms is Italian rye grass, which is used extensively for fodder as timothy is used in this country. This grass will stand large quantities of water; it is said to yield four good crops a year; and it is so vigor-

ous in growth that it keeps down weeds to a considerable extent. It exhausts the land in about three years, however, and must then be temporarily replaced by some other crop. In alternation with rye grass, mangolds (mangel-wurzel), which are beets of a coarse variety, are frequently grown, and are used as cattle food. Osier plantations often occupy a portion of the area, and cabbages of various sorts are frequently cultivated. In order to utilize the products of the farm, dairy cattle are often kept and stock-raising interests may even predominate. The luxuriant growth of weeds necessitates constant working over of the land; and at some of the larger plants steam cultivation is carried out. The direction of a large sewage farm is an administrative task of considerable magnitude.

The operation of sewage farms varies widely, according to the condition of the soil. In some places, as at Altrincham and Nottingham, sewage is discharged upon a given area for twelve hours out of the twenty-four. At Croydon each plot is dosed for a twenty-four hour period every three or four days. In most cases the dosing period is longer still. At Aldershot camp sewage is discharged upon one area continuously for six to ten days, and at Leicester for fourteen days. Whatever may be the dosing period, a corresponding rest must of course be given. Each particular plot is under irrigation for from one-fourth to one-half the time. To obtain an available area of proper soil is the chief problem in sewage farming. A porous soil absorbs a considerable volume of sewage and purifies it as it passes through its minute pores. An impervious soil either becomes quickly clogged or discharges crude sewage through cracks and fissures. A light soil, on a sandy or gravelly subsoil, proves most satisfactory. Peat, chalk, and clay, on the other hand, are bad; all three are too impervious and the last two are liable to dangerous cracking. With unsuitable soils rates of filtration must be low. Rideal (1906) estimates that the sewage from 100 persons can be treated on an acre of loamy gravel and that the number may rise to 500 under rarely favorable circumstances, while with stiff clay it falls to 25. The rates commonly in use in England vary from 2000 gallons per

acre per day at Leamington and Wrexham to 15,000 gallons at Cheltenham. The evidence collected by the Royal Commission (R. S. C., 1908) pointed to "a maximum rate of 30,000 gallons per acre, or 1000 persons per acre, with the best land, after preliminary treatment, although some witnesses put the rate as high as 60,000 gallons per acre, or 2000 persons per acre, under similar conditions. With unsuitable land, such as clay, not more than 3000 gallons per acre can be efficiently treated, even after settlement of the sewage."

In Germany, the process of sewage irrigation is carried out with greater precautions and, in general, with better results than in England. In the first place, no attempt is made, as in England, to treat sewage on clayey or peaty soils. Irrigation areas are of sand, and almost always carefully underdrained and provided with well-designed distribution systems. Studies at Berlin have shown that suspended solids, and particularly fatty materials, are highly detrimental to the process. Hence, careful provision is generally made for the removal of suspended solids. The famous sedimentation tanks at Dortmund were designed as preliminary to irrigation. In some of the German plants, sewage is actually distributed by hand, from hose-pipe nozzles. The rates in general use range from 2000 gallons per acre per day at Brunswick to 7000 at Dantzic, and probably average about 4000 (Bredtschneider and Thumm, 1904).

When an irrigation farm is overdosed it becomes "sewage sick," to use an expressive English term (Fig. 59). The surface clogs, pools are formed, putrefaction begins; only a complete rest with thorough plowing restores the health of the area. Under such conditions the temptation to discharge unpurified sewage into the nearest stream is very strong. At times of rain, when sewage flow is highest, the crops are least in need of water and may be seriously damaged by it. The aims of sewage purification are apt, under such conditions, to be sacrificed to those of agriculture. Thus, at the famous Craigentinny Meadows, where profitable crops are obtained from once barren areas of blown sands, we are told that "the great bulk of the foul water merely

runs over the surface of the ground and deposits a portion of its suspended matter" (Barwise, 1904). Many of the English farms, on the other hand, have been operated for thirty years so as to yield a satisfactory effluent without the production of any local nuisance.

FIG. 59. View of a "Sewage-sick" Irrigation Area (copied by permission from Dunbar, 1908).

Efficiency of Broad Irrigation. The extent of the purification effected by broad irrigation varies very widely, according to the nature of the available soil, the construction of the beds, and the method of operation. If beds of good sand are underdrained and operated as intermittent filtration areas, excellent results may be obtained. On the other hand, sewage which is allowed to run continuously, for long periods, over a clayey soil is not efficiently purified.

The statistics for eight of the principal English sewage farms have been compiled in the tables on pages 203-204 from the fourth report of the Royal Sewage Commission (R. S. C., 1904) and they give a fair idea of general practice:

TABLE XLIV

ENGINEERING DATA FOR EIGHT ENGLISH SEWAGE FARMS

	Aldershot camp.	Altrincham.	Cambridge.	Croydon.	Leicester.	Nottingham.	Rugby.	South Norwood.
Date of construc...	1864	1870	1895	1861	1891	1880	1867	1864
Tributary pop.....	20,000	18,000	50,000	100,000	197,000	258,584	6,000	21,000
Irrigable area, acres	120.5	35	74	420	1,350	651	35	152
Pop. per acre.....	166	514	676	238	146	397	171	138
Dry-weather flow, million gallons..	1	.8	2.25	4	7.25	7	.3	.6
D. W. F. per capita	50	44.4	45	40	36.8	27	50	28.5
D. W. F. per acre per day........	8,000	23,000	30,000	10,000	5,000	11,000	9,000	4,000
Rain fall, inches, per year..........	22	37	21	23.6	20	25	25.6	24
Storm water treated	Very little. Separate system.	Very little.	Small amount.	All from sewers.	2½ times D. W. F.	Very little.	Nearly all.	All from sewers.
Character of sewage.	Domestic.	Mainly domestic.	Mainly domestic.	Mainly domestic.	¼ trade.	$\frac{3}{7}$ trade.	Mainly domestic.	Domestic.
Preliminary treatment.	Screening and settling tanks.	Settling tanks.	Screening and settling tanks.	Screening.	Screening and settling tanks.	Part screened.	Screening and settling tanks.	Screening and settling tanks.
Land treatment...	Filtration.	Filtration.	Filtration.	Surface irrigation and filtration.	Surface irrigation and filtration.	Filtration.	Surface irrigation and filtration.	Surface irrigation and filtration.
Soil..............	Coarse sand	Peaty.	Sandy loam.	Gravelly loam.	Stiff clayey.	Sandy loam and gravel.	Clay.	Clay.
Subsoil...........	Very fine sand.	Sand and gravel.	Gravel and sand.	Gravel and sand.	Dense clay.	Gravel and sand.	Stiff clay.	London clay.
Ratio of D. W. F. to flow of river receiving effluent.	1 : 6	1 : 3	1 : 15	1 : 12	1 : 1	1: 160	1 : 30	3 1

TABLE XLV

ANALYTICAL DATA FOR EIGHT ENGLISH SEWAGE FARMS

Analyses. Parts per million.

	Aldershot camp.	Altrincham.	Cambridge.	Croydon.	Leicester.	Nottingham.	Rugby.	South Norwood.
Total suspended solids								
In sewage	366	720	265	345	341	519	473	219
Total organic nitrogen								
In sewage	50.6	12.4	18.8	20.7	23.4	31.5	32.9	14.9
In effluent	9.4	2.6	2.1	2.6	4.5	.8	1.7	10.4
Nitrogen as free ammonia								
In sewage	78.7	22.9	38.2	45.0	81.2	39.8	61.1	35.4
In effluent	18.5	12.3	6.5	14.8	16.5	1.3	16.2	8.7
Nitrogen as alb. ammonia								
In sewage	16.2	6.2	9.1	9.1	11.9	14.5	17.3	6.7
In effluent	2.6	1.3	.9	1.4	2.0	.3	1.8	1.0
Nitrogen as nitrites and nitrates								
In effluent	32.1	....	12.6	3.7	5.1	20.6	5.2	3.9
Oxygen consumed 4 hrs. at 80°								
In sewage	207.9	52.2	10.8	124.8	223.5	232.1	184.4	77.1
In effluent	27.2	17.5	8.1	14.1	25.0	1.9	14.1	14.4

The low rates on clayey soil at Leicester, Rugby, and South Norwood will be noticed, as well as the fact that careful screening and settling has in most cases been found a necessary preliminary. The analyses, which from their source may be considered representative, indicate that English irrigation effluents are by no means of exceptional quality. The Nottingham results are excellent, and those obtained at Cambridge, fair. The Aldershot plant appears to be doing good work, in view of the very strong

Fig. 60. Getting in the Hay Crop on an English Sewage Farm.

sewage with which it deals. At Altrincham, on the other hand, a weak sewage is not well purified, and the effluents obtained at Croydon, Leicester, Rugby and South Norwood can scarcely be considered satisfactory. The most important factor appears to be the nature of the soil, and its necessary corollary, the method of operation. At Aldershot, Cambridge and Nottingham, sandy areas are operated by downward filtration alone. Leicester, Rugby and South Norwood stand at the other extreme; their soils are mainly clay and the sewage is treated largely by surface irrigation.

In regard to organic stability, as determined by incubator tests, the experts of the Royal Commission reported that samples of the effluent at Nottingham never putrefied, while the Cambridge effluents also stood very high, and of the samples from Leicester and Aldershot 90 per cent gave no secondary putrefaction. Norwood, Croydon and Rugby, on the other hand, gave putrescible effluents about one-fourth of the time. On the whole, it seems fair to conclude from a general survey of English conditions that when a sufficient area of porous soil, with a low-water table, is available a well-managed irrigation area may yield effluents of considerable organic purity.

With regard to bacterial removal similar differences appear.

TABLE XLVI

BACTERIAL EFFICIENCY OF ENGLISH SEWAGE FARMS

(R. S. C., 1908.)

Bacteria per c.c. in effluent	Aldershot.	Altrincham.	Cambridge.	Croydon.	Leicester.	Rugby.	So. Norwood.
Gelatin, 20°	183,266	263,400	711,476	1,413,200	532,777	637,133	778,322
Per cent removal compared with sewage	99	99	94	95	95	97	98
Agar, 37°	37,308	7,275	78,327	112,000	70,500	81,526	35,157
Per cent removal compared with sewage	99	99	94	97	95	97	99
B. coli, approximate	1,000	100	1,000	1,000	1,000	1,000	100

The table above indicates the general results obtained in the studies of the Royal Commission. Average figures for Nottingham are not given, although it is stated that the total number of bacteria in the effluent was frequently less than 1000 per c.c. and that the removal of *B. coli* was usually very satisfactory. In general the table shows that the number of bacteria in the effluents is very high. In some cases even this high value represents a good per cent purification, as measured against the very strong sewages applied. In other cases, however, the per cent purification is low; for 94 and 95 per cent is a poor grade of puri-

fication when dealing with such large absolute numbers of bacteria as are found in sewage.

Sanitary Aspects of Sewage Farming. The success of any purification process, from a sanitary standpoint, must be judged in two ways. The final effluent produced should be of satisfactory quality, and conditions about the plant should be such that no serious local nuisance is created. In regard to the first point,

Fig. 61. Osier Bed on an English Sewage Farm.

it seems clear that the efficiency of broad irrigation depends entirely upon the nature of the soil and the method of operation. On the whole, it may be said that a sewage farm, under the best conditions, may yield a better effluent than can be obtained from contact beds or trickling filters. Where the soil is heavy, however, the results of irrigation are very much inferior to those which can be attained by the so-called "artificial" processes. Furthermore, in sewage farming there is always a tendency to the by-passing of surplus sewage at times of rain, and in many

instances this militates seriously against the general efficiency of the process.

In regard to the second point, there is no reason to anticipate any serious local nuisance from a properly operated irrigation area. Prior to the report of the Sewage of Towns Commission in 1858, there was a general fear that sewage spread out on the surface of the land would breed the miasms of disease. There is often an appreciable odor on the sewage farm itself, and disease germs might conceivably be spread by insects and in other ways from any sewage disposal area. On the Berlin farms, however, there is a resident population of four thousand persons, yet the careful observation of the German authorities has failed to show any detrimental influence upon health. It is of course obvious that well waters in the neighborhood of irrigation areas must be subject to strict supervision. Recent studies in England have shown that specific bacteria may pass for a distance of two miles in less than three days through chalky soil of a porous nature.

Finally, there is one other sanitary problem to be considered in connection with sewage farming, — the possibility that infection may be spread by the crops grown on the fields. Where vegetables are eaten raw, after irrigation with sewage there may, no doubt, be serious danger. An epidemic of 63 cases of typhoid fever at the Northampton (Mass.) Insane Hospital in 1899 was pretty clearly traced to celery manured with sewage sludge (Mass., 1900). The English experts, McGowan, Houston and Kershaw (R. S. C., 1904), would even go so far as to limit sewage farms to the raising of food for cattle: "We are, on the whole, not in favor of sewage farms being utilized for the raising of crops for human consumption."

This seems an extreme position. The cultivation of fruits and vegetables which grow near the ground and which are to be eaten raw should certainly be prohibited. With this restriction, however, there seems no reason why irrigation areas should not be cropped for the market. The long experience of Berlin and Paris indicates that there need be no serious

danger of the spread of disease from irrigated crops, under such conditions.

Economic Results of Sewage Farming. The economic advantages of sewage farming have long been a debated question. The utilization of waste products is always an attractive idea; and sewage fields covered with a rich mantle of luxuriant vegetation make a strong appeal to the imagination. It has been said, however, that it is poor economy to save something by a process which costs more than the value of what is to be saved. English chemists estimate the manurial value of sewage at from 1 to 4 cents a ton (Rafter and Baker, 1894). This value can no doubt in part be recovered, since the crops grown on sewage fields are often astonishingly heavy. Whether it really pays to recover it, however, is another question; and the answer entirely depends upon varying local conditions. In many English towns the operation of farms has proved unprofitable, and there is some tendency toward their abandonment. Lieut.-Col. A. S. Jones of the Aldershot farm and others are, however, ardent advocates of the process; and in certain cases the farm, besides paying all running expenses, yields in some years as much as $12 an acre toward rent (Baker, 1904).

The British Royal Commission in its final report (R. S. C., 1908) gives some careful estimates, which indicate pretty clearly the relative cost of land disposal under different conditions. The actual figures are somewhat high, according to American standards; but it must be remembered that the strength of English sewage makes it much harder to treat than our own. The increase in expense with poorer soils and the extent to which the returns from crops may be made to pay the cost of disposal are fairly indicated in the table below. The estimates are based on a flat price of $484 per acre for land and a laborer's wage of $5.04 per week. They include provision for preliminary sedimentation and sludge disposal, as well as for construction and operation of the beds themselves.

Comparative costs for percolating beds range from $19.74 to $24.28; and for contact beds, from $27.28 to $35.28.

TABLE XLVII

ESTIMATED COSTS AND RECEIPTS. SEWAGE FARMING IN ENGLAND

Per million gallons.

Nature of soil.	Process.	Gross cost.	Receipts.	Net cost.
Sandy loam overlying gravel and sand	Filtration with cropping	$16.45	$1.65	$14.80
Do.	Filtration, little cropping	12.02	.78	11.24
Do.	Surface irrigation with cropping	20.27	2.88	17.39
Heavy soil overlying clay	Surface irrigation with cropping	29.21	3.96	25.25
Stiff clayey soil over dense clay	Surface irrigation with cropping	41.47	6.62	34.85

There are two practical questions upon which these estimates throw light. The first problem which confronts any community contemplating sewage disposal is the alternative between land treatment and the so-called artificial processes (contact beds and percolating beds). The figures show that land treatment is cheaper where there is suitable sandy soil available, but that percolating beds are more economical than treatment on unsuitable land. Granting that land treatment is indicated, the next question is whether the area should be used for sewage disposal alone or whether the treatment of sewage should be combined with cropping. McGowan, Houston and Kershaw, in their valuable report to the Royal Sewage Commission, conclude:

"Although we are of opinion that sewage farms in general can never be expected to show a profit if interest on capital expenditure is included, the fact that in favorable seasons some of them more than cover the working expenses is a point in favor of cropping in connection with the land treatment of sewage" (R. S. C., 1904). On the other hand, according to the table of estimates cited above from the final report of the Royal Commission, the net cost of filtration with cropping is greater than that of filtration on similar land with little cropping.

The Sewage Farms of Paris and Berlin. The most notable examples of broad irrigation on a large scale are the sewage farms of Paris and Berlin. Irrigation was first suggested at Paris by Mille as far back as 1865. It met at first with much opposition on theoretical grounds; but after valuable preliminary investigations, to which reference has been made above, an experimental study was carried out in 1868, at Gennevilliers, in comparison with chemical precipitation. The land treatment proved entirely successful. When it came to the point of laying out sewage farms on a large scale, however, there developed a practical, political opposition on the part of the communities where the sewage was to be distributed, which was much more serious than the earlier scientific doubts of the efficacy of the process. This obstacle was not overcome until 1889, when a law finally passed the French parliament permitting the general development of the irrigation scheme. A new fight was made against loans for actual construction; and the farms were not extensively laid out until 1895 (Surveyor, 1900).

The Paris plant now consists of four large areas, the two oldest at Gennevilliers and Achères and two of more recent origin at Pierrelaye and Carrières-Triel. The total irrigated area amounted in 1905 to 13,597 acres, of which 4359 acres were owned by the city (Calmette, 1907). With 185,000,000 gallons a day, the net rate was about 12,000 gallons per acre. The land is for the most part of excellent quality, largely the ancient alluvium of the Seine. The sewage is carefully sedimented and screened at the Clichy pumping station. At the farms it is distributed from concrete outlet chambers by open ditches. At Gennevilliers, the main channels of masonry or concrete are 39 to 49 inches in diameter, and their smaller branches are 12 to 23 inches in diameter. On this farm alone there are 34 miles of distributing channels besides the final ramifications of the earth furrows. Concrete underdrains are laid 13 feet below the surface (Surveyor, 1900). About a third of the land, as noted above, is owned by the city, but most, even of this, is privately operated under contract. At Gennevilliers the land is privately owned and the

use of sewage is entirely voluntary, so that the demand for it is a fair indication of the success of the process. The city maintains, however, a model experimental garden in which various fruits and flowers are cultivated. On privately owned and operated land the responsibility of the city ceases at the outlet chambers. Large portions of the farms are used for pasturage; and among the crops grown are peas, artichokes, tomatoes, onions, potatoes, asparagus, sugar beets and cereals. The cultivation of strawberries, salad crops, and other foods which are freely exposed to the sewage and then eaten raw, is prohibited. It is an interesting commentary on the economic aspect of sewage farming that the city has recently had serious difficulty in securing extension of its farms on account of the opposition of farmers who objected to the unfair advantage given to the favored few who receive sewage for irrigation. At the Gennevilliers farm (of 2000 acres), the crop is worth over $400,000 a year. It is maintained by competent authorities (Calmette, 1907) that the farms as a whole could be operated far more efficiently than at present if the agricultural aspects of the case were more intelligently kept in view. Asparagus, for example, is a crop which is not at all adapted for intensive sewage treatment, and timber land is also wasteful on an irrigation area.

There are no published data by which the financial success of the Paris farms can properly be judged. The total cost of the farms up to 1900 was $7,220,000; and the annual operating expense nearly a million dollars a year (Surveyor, 1900). A part of this sum is made good by the sale of crops and the rent of irrigated land. At Gennevilliers the farmers use the sewage freely, and there is no return to the city at all. At the other farms the rent of the land is doubled or trebled by irrigation. The profits altogether are small, however, and the process as a whole a costly one (Calmette, 1907).

Of the whole volume of sewage applied to the Paris farms, fully one-half reappears in the drains; and regular analyses, made at the municipal observatory of Montsouris, show that the purification attained is excellent. The following figures are fairly typical:

TABLE XLVIII

ANALYSES OF SEWAGE AND EFFLUENTS AT PARIS

Parts per million. (Calmette, 1907.)

	Organic matter.	Nitrogen as —	
		Nitrates.	Ammonia.
Sewage. Clichy	43.3	.3	22.0
Effluent. Gennevilliers	1.0	31.1	.0
Achères	1.7	17.9	.5
Pierrelaye	.8	14.2	.0
Carrières-Triel	1.2	26.2	.0

Bacterial results are less satisfactory. Frequently the numbers are down to a few hundred per cubic centimeter; but at other times they may rise to hundreds of thousands. The most serious shortcoming of the Paris farms is, however, their inadequate capacity, which makes it impossible for them to handle the heavy sewage flows at the time of the spring rains. In such periods 35 per cent of the total sewage may be discharged, unpurified, into the Seine.

The Berlin sewage farms offer examples of broad irrigation under better conditions, for they are not only the largest, but perhaps the most efficiently managed in the world. The four farms at Osdorf, Falkenberg, Grossbeeren and Malchow were laid out between 1876 and 1882. Three more have since been added, at Sputendorf, Blankenfelde and Buch. There were altogether in 1907 (Berlin, 1907) about 39,000 acres of irrigated land, mostly alluvial sandy soil, of excellent quality. According to Rideal (1901), this area is one and a half times that of the city itself. The net rate of filtration is 3000 gallons per acre per day. About 12,000 acres are classed as unproductive. Of the remainder, 22,000 acres are farmed by the city and 5000 acres are leased. A quarter of the area operated by the authorities is devoted to pasturage, and about a third to the cultivation of cereals, of which winter rye and oats are the most important. Potatoes and beets are grown in considerable amounts and a wide variety of other crops in smaller proportions. Access to the markets is con-

venient for most of the farms, and dairies and distilleries are maintained for immediate utilization of the crops. Even fish ponds are made to yield a part of the revenue, and the drains on some of the farms have been successfully stocked with brook trout.

The Berlin irrigation system has been notably successful from the sanitary standpoint. Suspended solids and fats are carefully removed from the sewage before it is applied to the beds. The farms themselves are well kept and exhibited as show places for visitors; at the Blankenfelde and Buch areas playgrounds are maintained for the school children of the city. The chemical purification attained in the effluents from the farms is indicated in the table below. Results are continually being improved by the construction of sedimentation basins for the effluent and by the laying out of special areas for secondary filtration.

TABLE XLIX

COMPOSITION OF SEWAGE AND EFFLUENTS AT BERLIN

Parts per million. (Dunbar, 1908.)

	Total residue.	Loss on ignition.	Oxidizability, potassium permanganate.	Chlorine.	Ammonia and albuminoid ammonia.	Nitric acid and nitrous acid.
Sewage	978.4	285.2	333.7	283.8	99.5	......
Effluent	987.0	124.0	33.6	232.7	2.3	146.6
Per cent purification	......	56	90	10	98	......

The bacteria in the effluents averaged 34,000 in forty-six samples examined in 1906. Individual counts ranged from 15 to 417,300 (Berlin, 1907).

The total operating cost of the Berlin farms for 1906 was $700,000, of which about half was for salaries and wages. The farms are operated in part by convict labor; but the living expenses of the convicts are paid and every gang of ten is in charge of a high-priced guard, so that the net expense for labor is not materially reduced. It must always be remembered, however, that the German military discipline maintained makes possible

an efficiency and economy which could scarcely be hoped for under other conditions. The receipts for 1906 amounted to $750,000, showing a handsome profit by comparison with running expenses alone. The capital sum expended on the farms up to 1907 aggregated about $12,000,000, half for the first cost of the land, and half for preparation of the fields, construction of buildings, etc. The total amount of sewage treated is about 117 million gallons per day. On this basis the cost of operation amounts to $16.40 per million gallons and the receipts to $17.60 per million gallons, while the interest on the invested capital, at $3\frac{1}{2}$ per cent, is $9.80 per million gallons.

Sewage Farming in the Arid Regions. In arid regions like those found in the western part of the United States, the conditions for sewage farming are particularly favorable. Sewage farms were laid out in Wyoming, Colorado, and Nebraska before most of the eastern cities realized that there was a sewage disposal problem. As Mr. M. N. Baker points out (Baker, 1893), there were three reasons for the rapid advance of sewage irrigation in this region. The low stage of western waters during the hot dry season frequently made sewage discharged into them an unbearable nuisance, the people were familiar with the general practice of irrigation, and the great value of all available water naturally led to the application of the sewage to crops when any method of purification was necessary. In some of the western states the total annual rainfall is only 5–15 inches and there is no rain at all for eight months of the year. The water value of the sewage in these arid lands, when added to its manurial value, made the case for irrigation irresistible. As Professor C. G. Hyde of the University of California has recently written to one of the authors, "The scheme is deservedly popular in California, where it is a criminal waste to discharge sewage effluents into inland streams or even into the ocean."

The first of the western irrigation plants was laid out at Cheyenne, Wyo., in 1883. By 1890 plants were in operation at Colorado Springs, Colo., Helena, Montana, and Santa Rosa and Los Angeles, Cal. Greeley, Colo., Hastings, Neb., and Trinidad,

Colo., followed very soon after. The experience of Los Angeles is of considerable interest, as an example of temporary success followed by embarrassment, due to economic changes and the concentration of population. Prior to 1889 the sewage from the city, amounting to seven million gallons, was carried to the so-called Vernon District, where it was taken by the South Side Irrigation Company and distributed to adjacent farms. So useful did the sewage prove that the value of the land rose from $2.50 an acre to from $15 to $25. Later, however, a boom set in, house lots were developed, and the population so increased that the sewage, which had helped to build up the district, became a nuisance and had to be taken elsewhere. Recently, a large outfall sewer has been constructed at great expense to discharge into the ocean.

In general, sewage farming has proved successful in the West, and it is likely to be the prevailing method of the future in such regions. At present a score or more of farms are in operation. Salt Lake City, Utah, Hastings, Neb., and Pasadena, Cal., are among the largest and best known. In California alone there are eight plants now in active use; at Bakersfield, Fresno, Pasadena, Pomona, Redlands, Salinas, Santa Rosa and Visalia. In some cases the land is owned or leased and operated by the town; in other instances the sewage is distributed on private land, with or without the payment of a bonus. The crops cultivated are those usual in the respective localities, — peas, beans, tomatoes, corn, cabbages, turnips, grass, alfalfa and fruit trees. In the more recent plants, as at Santa Rosa, Pomona and Fresno, Cal., a septic tank has been installed for preliminary treatment of the sewage. In many other cases it must be acknowledged that neither the construction nor the operation of the farms is especially calculated to secure prompt and inoffensive oxidation of organic matter. "The sanitary question of disposing of the sewage seems to be quite incidental" (Fuller, 1905).

The Pasadena sewage farm may be taken as a type of broad irrigation at its best, from the economic standpoint. Three hundred acres of land were purchased by the city in 1887, and after a long period of litigation in regard to rights of way for an outfall

sewer, the application of sewage was at last begun in 1892. The farm is about four miles from the city and is of good sandy loam. It is divided into fields one hundred feet in width and two hundred to four hundred feet in length, extending out on either side of main carriers or irrigation ditches. To irrigate the fields a dam of earth or of redwood board is inserted in the main carrier at the

FIG. 62. Cornfield on the Pasadena Sewage Farm.

lower end of a field and the sewage thus diverted into small furrows. The following details of operation are taken from the town report for 1903–04 (Pasadena, 1904):

"The first crops raised consisted of barley and wheat hay, pumpkins, corn and alfalfa, all of which are now successfully raised, except alfalfa, which, being a matted low-growing crop, the roots of which incline to sod, and cultivation being impossible, the solids of the sewage would collect in it on the side of the field next to the head ditch and the results were unsatisfactory conditions. It was finally decided to abandon the raising of

alfalfa although the yields were extraordinary and the profits excellent.*

"In 1892 sixty acres on the south end of the farm were planted to English walnuts. The trees having made such splendid growth, an additional thirty acres were planted in 1898, and last spring another twenty-seven acres were planted, making one hundred and seventeen acres in walnuts. During the first few years corn and pumpkins are raised in the walnut groves, but as the

FIG. 63. Walnuts on the Pasadena Sewage Farm.

groves come into bearing no other crops are raised. The sewage is turned into the bearing groves as soon as the leaves are well off the trees, which usually is about December 1st. It is not safe to unduly saturate the ground while the trees are in foliage, as a heavy wind, from which the locality is unusually free, would be disastrous to the trees.

"The sewage is kept in the walnut groves until the foliage comes out on the trees about April 1st, the entire flow for the

* With preliminary septic treatment alfalfa has proved a very satisfactory crop at other California farms.

past three or four winters having been disposed of in the groves. After the sewage is taken from the groves about April 1st, it is not turned back again until the following December. From April to December the sewage is used in the open fields. At times during haying or walnut-picking, when all of the farm help is needed to care for the crops, neighbors take the sewage on their own farms, the opportunity of which they eagerly avail themselves.

"Whether being used in the groves or in the open fields, the sewage is not allowed to run upon any one area longer than from four to ten days at a time, after which period it is turned upon another area, and as soon as the area upon which it had been running is sufficiently dry to be worked, which is usually within two or three days, it is thoroughly cultivated or plowed. Frequently the top surface is plowed under, but this is not done after every period of irrigation, as only thorough stirring with a cultivator is necessary. The number of days that the sewage can be allowed to run upon a given area at any one period depends upon the weather and atmospheric conditions. If the weather is hot and dry, the period reaches its maximum, but if the weather is damp and the atmospheric pressure low, when such odors as are present cannot rise readily but are held down close to the surface of the ground, the period is the minimum."

The average flow of sewage treated on the Pasadena farm in 1903–04 was 840,000 gallons per day. On the 300 acres of land in use, this is equivalent to a rate of 2800 gallons per acre.

The operating expenses of the farm for this year were $6310.91, mainly for labor. The revenues amounted to $11,643.57, of which $7847.29 came from the sale of walnuts. The land originally purchased in 1887 cost $125 an acre, or about $40,000 in all. In 1904 one hundred and sixty acres more were purchased at $150 an acre. Interest at 3½ per cent on the cost of the three hundred acres in use in 1903–04 would amount to $1400. Even taking this into account, the Pasadena farm is clearly a profitable undertaking.

The General Outlook for Sewage Farming. The availability of broad irrigation, as a practical method of sewage treatment, obviously depends to a high degree on varying local conditions. In dry countries, where every drop of water is precious, there can be no doubt that it is the ideal procedure. In California and

similar regions of the United States it is likely to be the prevailing method of sewage treatment. In India the rice fields at Madras and elsewhere are irrigated with sewage with marked success. On the other hand, it seems quite as certain that sewage farming on heavy lands is a mistake. The English communities, which have clung to sewage farming under adverse natural conditions, have demonstrated that it may be a failure, and a costly one.

Between these two extremes are cases in which the conclusion is less clear. With fair soil and not too heavy rainfall, broad irrigation may be operated satisfactorily by cities having at their doors large areas of cheap and infertile sandy soil. Its economic value, then, depends upon a number of minor variables. The cost of land, the cost of labor, the available markets, and above all the skillful management devoted to the farms, chiefly control the final result. Where all these conditions are favorable, broad irrigation may offer an economical method of sewage disposal, as demonstrated at Aldershot and at Berlin. Where all or any of these conditions are against the process, its success becomes more dubious. In England the general tendency is toward a decrease rather than an increase in the number of sewage farms. Calmette (1907), after a careful discussion of conditions on the Paris farms, concludes that the practice of broad irrigation is likely to be more and more restricted and ultimately abandoned, and Dunbar (1908) believes that even at Berlin artificial filters will one day take the place of sewage farms.

In any case, where sewage farms are to be maintained, the danger that sanitary efficiency may be sacrificed to economic success must be carefully guarded against. There is an inevitable antithesis between the agricultural and the sanitary requirements of broad irrigation, and only a large plant can reasonably hope to hold a superintendent competent to harmonize both aspects of the problem.

In the eastern United States natural conditions of soil and rainfall are generally more favorable to sewage farming than those which prevail in England. Economic and political conditions,

however, are against the process. The excessive water consumption in this country makes the per capita flow of sewage two or three times as high as it is in Europe. The labor charges in the United States would be twice as high as in England; and municipal politics seriously militate against efficiency of operation, though this latter objection is a temporary rather than a permanent one. Sewage farming is not likely to be one of the future activities of the American city, except in the arid regions.

CHAPTER IX

PURIFICATION OF SEWAGE BY INTERMITTENT FILTRATION THROUGH SAND

Early Studies of the Principles of Bacterial Purification. Dilution in water and broad irrigation on land are the two primitive and natural methods of sewage disposal. Both systems were at first developed from a purely empirical standpoint, and without any idea that the fundamental process, in either case, was the oxidation of organic matter under the influence of bacterial life. A few investigators, however, grasped the essential principles involved at an early date in the history of the art. Thus, about 1865, Alexander Mueller, City Chemist of Berlin, described the purification of sewage as a process of digestion and mineralization carried out by minute animal and vegetable organisms. In 1878 he took out a patent for a "process for the disinfection, purification, and utilization of sewage by the scientific cultivation of yeastlike organisms." In 1877 the fact that the purification of sewage is due to living organisms was demonstrated by Schloesing and Müntz in a series of experiments, in which it was shown that nitrification did not occur in soils sterilized by heat or chloroform (Schloesing and Müntz, 1877). Warington, in England, communicated to the Society of Arts, in 1882, a paper in which he pointed out that dilute solutions of ammonium salts or of urine would not nitrify when sterilized by boiling and supplied with air filtered through cotton. If, however, a small particle of fresh soil was added, active nitrification would set in. He further found that this process went on best in the dark, in the presence of an alkaline base, such as lime, and at a temperature of 5°–50° C. He adds, "Though, however, porosity is by no means essential to the nitrifying power of a soil, it is undoubtedly a condition having a very

favorable influence on the rapidity of the process; a porous soil of open texture will present an immense surface, covered with oxidizing organisms, and generally well supplied with the air requisite for the discharge of their functions."

Beginnings of Intermittent Filtration in England. The practical development of sewage purification, along intensive lines, and by the application of scientific principles, had begun even earlier than this with a series of significant experiments carried out by Sir Edward Frankland in connection with the first report of the Rivers Pollution Commission of Great Britain. In the early days of irrigation it was generally thought that the green plants played an essential part in the purification process. When the experts of the Commission visited Ealing and Chorley, they found that sewage was being treated with more or less success on uncropped land. This suggested the importance of a careful study of the purifying process, and Frankland at once began a series of experiments in his laboratory at London. Glass cylinders six feet high and ten inches in diameter were filled with various filtering materials, — gravel, sand, loam, and peat, — and London sewage was applied in various amounts, twice daily, for a period of four months. The results, on the whole, were highly satisfactory, and the Commission concluded that the sewage of 1000 persons could be treated on an acre of properly prepared sandy soil. Sewage was filtered, in Frankland's experiments, both upward and downward through various soils at different rates, and it was shown that a good effluent could be obtained by downward filtration through coarse gravel at a rate of 80,000 gallons per acre per day, while upward filtration produced only a foul and turbid effluent. Doubling the rate interfered with the purification, and it was noted that a resting or aerating period between the applications of sewage was a necessity. The principle of the process as a chemical oxidation of organic matter to water, carbon dioxide, and nitrates was clearly recognized, as well as the practical necessity for intermittency in operation. The cycles in the life of a sewage filter were picturesquely described in the following passage: "The conclusions arrived at

may be thus summarized: Sewage traversing a porous and finely divided soil undergoes a process to some extent analogous to that experienced by blood in passing through the lungs in the act of breathing. A field of porous soil, irrigated intermittently, virtually performs an act of respiration, copying on an enormous scale the lung action of a breathing animal, for it is alternately receiving and expiring air, and thus dealing as an oxidizing agent with the filthy fluid which is passing through it. The action of the earth as a means of filtration must not be considered as merely mechanical; it is chemical, for the results of filtration properly conducted are the oxidation, and thereby the transformation, of the offensive organic substances, in solution in the sewage, into fertilizing matters, which remain in the soil, and into certain harmless inorganic salts, which pass off in the effluent water" (Rivers Pollution Commission, 1870).

These researches of the Royal Commission indicated that with suitable soil the process of broad irrigation might be made intensive, the growing of crops being subordinated to the treatment of sewage at a more rapid rate. This principle was quickly applied on a practical scale by J. Bailey-Denton, who constructed an intermittent filter at Merthyr Tydvil, Wales, in 1871. About twenty acres of gravelly land, overlayed with loam, were laid out in four beds, carefully underdrained at a depth of five to seven feet. The surface of the beds was furrowed and cropped, but they were operated in accordance with Frankland's recommendations as true intermittent filters. The sewage was applied to each bed for six hours out of the twenty-four and the net rate was about 60,000 gallons per acre per day. This rate was later reduced to 16,000 gallons per acre by the addition of more land, and the plant has since been operated more as an irrigation area (Harvey, 1908). The original plant worked admirably, however, and was of importance as a demonstration of Frankland's process on a practical scale (Bailey-Denton, 1882).

Bailey-Denton installed other plants in England and Scotland upon the same principles; but as a rule English engineers were not impressed with Frankland's experiments. The conception

of intermittent filtration was merged and lost in the prevalent practice of broad irrigation and the intensive biological purification of sewage made no appreciable headway. As in so many other instances, the theoretical comprehension of a process was by no means the same thing as its demonstration in such a convincing form as to insure general acceptance. The real introduction into practice, of modern scientific methods of sewage treatment remained for American investigators. English sanitarians have generously recognized that "it was primarily due to the Massachusetts State Board of Health, who began their investigations in November, 1887, and have continued them ever since, that the bacterial treatment of sewage has been forced on public attention" (Watson, 1903).

The Lawrence Experiment Station of the Massachusetts State Board of Health. By the year 1880 the eastern part of the state of Massachusetts had become so thickly settled that the problems of stream pollution and sewage disposal began to press for solution. The same conditions which had been met in England twenty years before were beginning to be faced in the United States. Accordingly, in 1881 and 1884, special commissions were appointed to consider the pollution and protection of the streams in the vicinity of Boston. The second of these bodies made a report in 1886 which contained a review of sewage disposal practice in England and on the Continent and an earnest recommendation for a permanent commission intrusted with the duty of protecting the purity of inland waters. The paragraph in which the functions and the policies of such a body are outlined is so admirably expressed that it may well be quoted in full:

"Let these guardians of inland waters be charged to acquaint themselves with the actual condition of all waters within the state as respects their pollution or purity, and to inform themselves particularly as to the relation which that condition bears to the health and well-being of any part of the people of the commonwealth. Let them do away, as far as possible, with all remediable pollution, and use every means in their power to prevent further vitiation. Let them make it their business to advise and assist cities or towns desiring a supply of water or a system of sewerage.

They shall put themselves at the disposal of manufacturers and others using rivers, streams, or ponds, or in any way misusing them, to suggest the best means of minimizing the amount of dirt in their effluent, and to experiment upon methods of reducing or avoiding pollution. They shall warn the persistent violator of all reasonable regulation in the management of water, of the consequences of his acts. In a word, it shall be their especial function to guard the public interest and the public health in its relation with water, whether pure or defiled, with the ultimate hope, which must never be abandoned, that sooner or later ways may be found to redeem and preserve all the waters of the State" (Massachusetts, 1886).

In accordance with these recommendations, the Legislature of 1886 reorganized the State Board of Health and charged it with the duties outlined by the commission, namely, with the advice of cities and towns, corporations and individuals, as to water supply and sewage disposal, and ordered it to collect information and conduct experiments on the purification of sewage. The board promptly presented plans for extensive studies of the fundamental principles involved in sewage treatment and asked and obtained a considerable appropriation for its work.

Hiram F. Mills, a distinguished hydraulic engineer and a member of the board, organized the investigation, with the assistance of Professors T. M. Drown and W. T. Sedgwick, both of the Massachusetts Institute of Technology, as chemist and biologist, respectively. An experiment station (Fig. 64) was fitted up in 1887 on the bank of the Merrimac River at Lawrence, under the immediate charge of Mr. Allen Hazen. Here ten circular cypress tanks, 17 feet in diameter and 6 feet deep, were filled with various filtering materials — sand, gravel, peat, river silt, loam, garden soil, and clay — and dosed with sewage pumped from the city sewer. From most of the filters, effluents of a high degree of purity were obtained; and the tanks themselves remained clean and sweet, confounding the bystanders, who predicted that "in a fortnight, at the latest, the filters would become clogged and foul, and the whole neighborhood pestilential" (Sedgwick, 1902).

One condition, however, was essential to success,—the application of the sewage intermittently, in small doses, so that the air supply necessary for its oxidation should be available. With porous sand and gravel the proper balance of sewage and oxygen was easily maintained. A layer of sewage an inch in depth applied each day to such a bed of sand spreads itself through a layer of sand about nine inches in depth and there rests in thin films on

FIG. 64. Experimental Filters at the Lawrence Experiment Station (copied by permission from Henneking, 1909).

the surfaces of the sand grains, in intimate contact with about twice its own volume of air. Under these conditions the sewage is rapidly converted into a clear and well-nitrified effluent. With more impervious soils it appeared almost impossible to preserve this oxygen supply, since capillarity prevented them from ever draining dry. Thus with peat and garden soil, even when operated at very low rates, clogging occurred and nitrification failed. All the other filters showed good purification at rates of from 20,000 to 100,000 gallons per acre per day, the chemical qual-

ity of the effluents being equal in many cases to that of well waters in use in the city of Lawrence.

The true nature of sewage purification as a bacterial oxidation was clearly brought out in these experiments. Intermittency of application supplies the needed oxygen, and any fairly porous material will serve as a resting place for the active bacteria.

"The experiments with gravelstones give us the best illustration of the essential character of intermittent filtration of sewage. In these, without straining the sewage sufficiently to remove even the coarser suspended particles, the slow movement of the liquid in thin films over the surface of the stones, with air in contact, caused to be removed for some months 97 per cent of the organic nitrogenous matter, a large part of which was in solution, as well as 99 per cent of the bacteria, which were of course in suspension, and enabled these organic matters to be oxidized or burned, so that there remained in the effluent but 3 per cent of the decomposable organic matter of the sewage, the remainder being converted into harmless mineral matter.

"The mechanical separation of any part of the sewage by straining through sand is but an incident, which, under some conditions, favorably modifies the result; but the essential conditions are very slow motion of very thin films of liquid over the surface of particles having spaces between them sufficient to allow air to be continually in contact with the films of liquid.

"With these conditions it is essential that certain bacteria should be present to aid in the process of nitrification. These, we have found, come in the sewage at all times of the year, and the conditions just mentioned appear to be most favorable for their efficient action and at the same time most destructive to them and to all kinds of bacteria that are in sewage" (Mills, 1890).

The Lawrence work had extended over a period of two full years when the first report was published in 1890. The experimental filters were of sufficient size ($\frac{1}{200}$ of an acre each) to warrant the application of their results on a practical scale. They had been operated out of doors, in all weathers, and at all seasons. The effect of these early experiments was, therefore, to demonstrate beyond a doubt that intermittent filtration was a valuable working method for the purification of sewage.

One of the first results of the Lawrence experiments was the construction at Framingham, under the supervision of Simpson C. Heald, a young Massachusetts sanitary engineer, of an intermittent filtration area for the treatment of the town sewage. The sewage, which in times of dry-weather flow amounted to 650,000 gallons per day, was run to two reservoirs (each with a capacity of 431,000 gallons) and then pumped to a filtration area of about twenty acres. This plant was constructed in 1889, and has been operated ever since 1890 with excellent results. Other communities quickly followed suit. Gardner and Marlboro constructed filtration plants in 1891 and at the same time the first filter in Connecticut was built at Meriden.

General Principles of Intermittent Filtration. Intermittent filtration differs from broad irrigation as a controlled scientific process differs from a merely empirical one. Instead of pouring sewage over any convenient plot of land, specially selected areas of sand or gravel of proper fineness and evenness are used. Instead of allowing the sewage to find its way over or through the land as best it may, the beds are carefully underdrained so that it shall filter through a thickness of four or five feet of aerated sand. The application of the sewage is so regulated by intermittent dosing that the bed shall never become waterlogged so as to cut off its air supply. As compared with broad irrigation the volume of sewage treated per unit area was increased tenfold by the regulated intermittent process (50,000–100,000 gallons per acre per day against 5000–10,000).

The most important result of the Massachusetts experiments was, however, not merely the discovery that intermittent filtration offered an excellent method of sewage disposal for communities having available sand deposits in their neighborhood; it was rather the clearing up and making definite of the essential nature of the sewage purification process itself. This process had been more or less clearly understood by Mueller, Schloesing and Müntz, Warington and Frankland, but its details were elaborated and its practical importance convincingly proved for the first time by the Lawrence workers. Their greatest contribution to

the art was the demonstration, for the first time, of the fundamental fact that sewage purification is a slow burning or combustion of organic matter, carried out by living micro-organisms growing on the surface of a porous substratum and working only in the presence of abundance of oxygen. The process of intermittent filtration itself is only suitable for certain fortunate communities; but the general principles established at Lawrence underlie all newer processes of sewage treatment of any type whatsoever. The contact bed and the trickling filter are modified and improved devices for applying the same laws worked out in their elementary form in the little testing laboratory on the shore of the Merrimac.

The Chemistry of Intermittent Filtration. The chemical changes which take place in the sand filter are, so far as we now understand them, relatively simple ones. They consist in essence of a more or less direct oxidation of organic matter, the nitrogen finally appearing in the form of nitric acid and the carbon and hydrogen as carbonic acid and water. There appears, however, always to be an intermediate stage in which nitrous acid is formed from the organic nitrogen, to be later oxidized to the nitric form.

The nitrous and nitric acids are present, of course, in combination with the alkali metals and the alkaline earths, while the carbonic acid is partly combined and partly escapes as gas. According to Adeney (Letts and Adeney, 1908), the first of these two stages, nitrosification, is further separable into two, at least when the oxidation takes place in water. He finds that " the organic matters first suffer complete fermentation, the products of change being carbonic acid, water, ammonia, and organic substances possessing the chemical and physical properties of the humus of cultivated soils, and of the organic substances to be found in all unpolluted natural waters, especially in those of upland surfaces. A second stage of change subsequently ensues, in which these humus matters and the ammonium compounds are further fermented, the resulting products being carbonic acid, nitric acid, and water. The inorganic products of the first stage of fermentation result from the action of the respiratory process,

and possibly also of enzymes; and the humus-like substances are waste products from the synthetical processes."

Measured by the ordinary methods of sanitary chemical analysis, these changes are manifested by the disappearance of free and albuminoid ammonia and the formation of nitrites and nitrates. Under favorable conditions the transformation of nitrogen to the mineral form may be almost quantitative. The classic experiment of Scott-Moncrieff at Ashtead, in 1898, furnishes an excellent illustration of the nature of the process, although his filtering material was of coke, like that used in trickling filters, and was not therefore exactly comparable with sand. He constructed a series of nine trays of 1-inch coke, each 2 by 7 feet by 7 inches deep, arranged one over the other, with a space of 2 inches between each pair. The effluent from a "cultivation tank" was discharged on the upper tray by tipping buckets at a rate of 1,300,000 gallons per acre per day (140,000 on the whole area of nine trays), and its passage through the series occupied from eight to ten minutes (Fig. 35). The results of the gradual purification are indicated in the table on page 232, and the steady decrease of organic matter, with a corresponding increase of nitrates and a temporary formation of nitrites, will be observed. There was a total loss of a quarter or more of the nitrogen in this experiment; but in some similar work by Rolants and Gallemand (1901) it was shown that 297.9 mg. of nitrogen out of 301.8 mg. applied could be recovered at the end of the process in the nitric form.

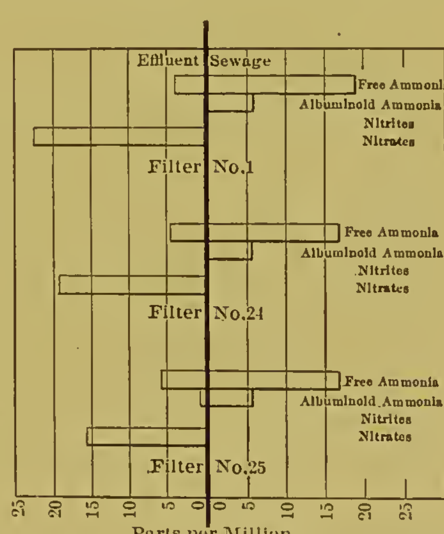


FIG. 65. Nitrogen changes in Intermittent Filtration at the Technology Experiment Station.

The general course of the changes in the passage of sewage through a sand filter is indicated graphically in Fig. 65, from

TABLE L

RESULTS OF TRICKLING FILTRATION THROUGH SCOTT-MONCRIEFF'S TRAYS

Parts per million. (Scott-Moncrieff, 1899.)

Effluents of —	Nitrogen as —				Oxygen consumed in 4 hours at 80° F.
	Free ammonia.	Albuminoid ammonia.	Nitrites.	Nitrates.	
Cultivation tank	103.0	12.3	0.0	1.2	98.4
First tray	86.5	10.3	9.9	1.0	66.9
Second tray	74.2	8.2	9.0	4.8	57.7
Third tray	41.2	4.9	7.8	18.7	44.9
Fourth tray	33.0	2.9	6.6	27.6	17.3
Fifth tray	12.4	1.2	4.8	46.8	12.8
Sixth tray	14.4	2.9	5.1	44.2	15.0
Seventh tray	2.9	2.5	0.0	66.0	7.6
Eighth tray	1.7	5.3	0.0	73.2	4.0
Ninth tray	2.1	4.9	Slight tr.	90.0	5.9

results obtained at the Sewage Experiment Station of the Massachusetts Institute of Technology (Winslow and Phelps, 1906).

The Bacteriology of Intermittent Filtration. Pasteur expressed the conviction in 1862 that the process of nitrification would eventually be shown to be due to the activity of living microorganisms. This suggestion was verified by Schloesing and Müntz in 1877. They prepared experimental filters of quartz sand mixed with crushed limestone and passed sewage through for a considerable time. At first there was no change, but gradually nitrification set in and the filters finally became very active. When exposed to the vapors of chloroform the process stopped, and after the removal of the chloroform nitrification soon began again, if the filter was seeded with earth from a nitrifying field. Heating to 100° was also shown by these investigators to stop all action, and it was thus proven that living organisms of some sort were the cause of this particular fermentation (Schloesing and Müntz, 1877). Warington and the Franklands and many other observers extended and confirmed these results; but for a long time all attempts to isolate the particular organisms concerned seemed doomed to failure. Finally the Franklands,

by the dilution method, succeeded in isolating from a nitrifying solution a short, stout bacillus which formed nitrites actively but which refused to grow on gelatin media (Frankland, 1890). In the same year, Winogradsky (1890) independently discovered the same organism, which he called *Nitrosomonas*, growing it in solutions containing ammonium sulphate, potassium phosphate and basic magnesium carbonate without organic material. In the next year Winogradsky (1891) showed that the change from nitrous to nitric acid is dependent upon the presence of another peculiar group of organisms (*Nitrobacter*), which is also difficult to cultivate on ordinary media and which, working in symbiosis with *Nitrosomonas*, is able to carry on the complete nitrifying change. In this year, too, he successfully cultivated these organisms on solid media made up of inorganic constituents stiffened with silica jelly. Richards and Jordan (1890) in the United States, and Warington (1891) in England, promptly confirmed these results. An important series of investigations of the nitrifying organisms was later reported by Winogradsky and his colleagues (Winogradsky and Omelianski, 1899 and Omelianski, 1899), from which it appeared that the *Nitrosomonas* acts only upon free ammonia and is unable to attack more complex organic bodies. The failure to find the nitrifying bacteria prior to 1890 was due simply to the unsuitability of the rich organic media employed. In recent years they have been isolated and studied by a large number of workers, of whom the most important are perhaps Schultz-Schultzenstein (1903), Boullanger and Massol (1903), Wimmer (1904), Calmette (1905) and Chick (1906).

The nitrite organisms, grouped under the name *Nitrosomonas*, differ more or less in soils from various parts of the world. They are all, however, oval bacilli which grow in compact zoöglea masses; some forms have a motile stage and others have not. In the process of their growth they apparently utilize the energy of the ammonia compounds to break up the carbonic acid molecule, from which they obtain their supply of carbon. Substances like urea, asparagin and egg-albumin check their activity, and even food material as close to ammonia as the amines cannot be attacked

It is clear, therefore, that in nature the *Nitrosomonas* organisms are dependent on a sort of symbiosis with the ordinary putrefactive bacteria which set free their food supply, ammonia, from its organic compounds. It appears probable, too, that the nitrite organisms only thrive well in the presence of a porous inorganic substratum on which they can form their zoöglea films. Stevens has recently shown, using both pure and mixed cultures, that nitrification is far more active in a partially dry soil than in a saturated soil or liquid, however well aerated.

The nitrate formers of the genus *Nitrobacter* are smaller oval rods, capsulated, and responding with difficulty to ordinary bacterial stains. They grow on agar media better than the nitrite formers, though with extreme slowness; but a very small amount of free ammonia checks their development. The nitrate formers are therefore dependent upon the nitrite formers for protection against ammonia, as the nitrite formers are dependent on the putrefactive bacteria for the formation of ammonia from more complex bodies. An interesting point, brought out by Beddoes (1899) and Adeney (Letts and Adeney, 1908), is the favorable effect exerted upon the activity of *Nitrobacter* by the presence of humus-like bodies.

The main practical requirements for the whole process of nitrification, as worked out on the basis of these bacteriological studies, may be stated somewhat as follows. A porous substratum must be provided for the growth of the active organisms. They must have an abundant supply of oxygen for their specific fermentations. The applied sewage must not be too strong; *Nitrosomonas* development is checked by the presence of .05 per cent of free ammonia. An alkalin base must be provided (as agriculturalists have known since the days of Varro), to unite with the nitric acid produced, since free acids, in a strength of .5 per cent, quickly stop the nitrifying process. As a rule, however, the alkalinity of ordinary domestic sewage is sufficient for this purpose. Nor is the particular base present an indifferent matter; the nitrites and nitrates of calcium and magnesium are less harmful than the corresponding salts of sodium and

potassium; .1–.5 per cent of sodium nitrate retards nitrosification as against 1 per cent of calcium nitrate. In any case, free drainage to remove the end products of the reaction must be provided. Finally, a marked excess of alkali may be detrimental. Burton sewage containing 100 parts of free lime would not nitrify till it was neutralized (Barwise, 1904). Rideal also notes a harmful effect due to carbon dioxide, and Letts believes that sodium chloride from sea water hinders the formation of nitrates at Belfast (R. S. C., 1902). The optimum temperature for the process of nitrification lies between 28° and 37° C. The *Nitrosomonas* forms are destroyed at 45° and the *Nitrobacter* group at 55°.

A sewage filter is really a device for cultivating the *Nitrosomonas* and *Nitrobacter* organisms under the conditions most favorable to their maximum activity. Like any other biological mechanism, it requires time to bring it to perfection. In the original Lawrence experiments it was found that nitrification did not start in new filters during the cold weather of winter. The delicacy of the reactions involved is shown very clearly in the trickling filter which responds even more readily to external changes. Fig. 87, in Chapter XI, is taken from the report of the Technology experiments with two eight-foot beds of broken stone. For the first six months of the life of the beds no important nitrification occurred. With the warm weather of spring, nitrites began to appear, reaching a maximum in September; and as the nitrites reached their height the nitrate formation began, and continued with increasing vigor during the whole of the following winter.

Construction of Intermittent Filters. The construction of intermittent sand filters in regions like Massachusetts is extremely simple. This particular part of the United States is covered with deposits of glacial drift, so that large areas of fairly level sandy soil are of very common occurrence, and all that it is necessary to do is to strip off the surface soil, usually not more than 1 foot in depth, to level off the sand area, divide it into beds by embankments made of the strippings, remove from the sand

beds so made any pockets of clay or quicksand, underdrain the beds, — best by digging trenches 4 feet to 6 feet deep, about 50 feet apart, the first being 25 feet from the edge of each bed, and placing in these trenches vitrified clay pipes, laid with open joints. These drains are connected with the main drains, placed in the embankments between the beds, and the sewage is brought

Fig. 66. General View of Intermittent Filter Beds at Brockton, Mass. (courtesy of G. E. Bolling).

to the plant by gravity or by pumping, and by various methods distributed upon the surface of the beds. In a few recent plants the filters have been surrounded by thin walls of concrete.

In certain cases the sandbeds have to be made by excavating the soil to the depth of 4 to 5 feet, and replacing it by sand taken from some neighboring knoll. Outside of the northeastern states it is frequently necessary to obtain suitable sand from considerable distances. The proximity of an adequate supply of sand

becomes, then, the controlling factor in determining the availability of this method of sewage purification. There is a compensating advantage, however, in the fact that the beds under these conditions can be built of the very best grade of material. The character of a particular deposit is determined, as in the study of sand to be used for water filtration (Hazen, 1893, Hazen, 1900). Test pits are dug in various parts of the sand bed and representative samples of one hundred grams or more are collected for examination. Each sample is then passed, with vigorous shaking, through a series of sieves of mesh ranging from .1 mm. to 5. mm., or thereabouts. The sand remaining on each sieve is weighed and a curve plotted using the size of each mesh in terms of millimeters as ordinate and the per cent of the whole sample by weight which passed that sieve as abscissa. The character by which any sand shall be judged is taken as that size, such that 10 per cent of the particles by weight are smaller, and 90 per cent larger, than itself. This is known as the Effective Size, and can be obtained directly from the plotting.

For sewage filtration the limits within which this value may vary are much wider than for water filtration; but the effective size should be not less than 0.25 mm. Furthermore, it is important for the best results that the sand should be fairly uniform and not stratified horizontally to any great extent. As Fuller (1904) has pointed out: "If fine sand, loam or clay remains above coarse sand, the latter is of limited benefit for filtration, because a water seal is formed at the bottom of the fine layer, due to the liquid held in the pores by capillarity, and air is excluded from entering the sand. If coarse sand overlies fine material near the surface, clogging sooner or later takes place at the junction and air is similarly excluded."

The size of individual beds at different filtration areas varies from .05 of an acre up to 1 acre, the very small beds being in towns where the daily flow of sewage is such that it would be impossible to use large beds. Thus, at Leicester, Mass., where the total flow of sewage is less than 30,000 gallons per day, there are

eight beds, each having a superficial area of 0.045 of an acre; while Andover, with 125,000 gallons of sewage per day, has twenty beds, each 0.18 of an acre; Marlboro, with 1,100,000 gallons of sewage, has eighteen beds, each 0.6 of an acre; Framingham, with 650,000 gallons of sewage, has eighteen beds, each a little over 1 acre in area.

In some places the beds are perfectly level, as is the case at Marlboro, Andover, and Pittsfield, while at other places the beds

FIG. 67. Furrows and Sewage Distributors at Brockton, Mass. (courtesy of G. E. Bolling).

are ridged, having furrows from 8 inches to 1 foot in depth. This difference in the surface of the beds is due chiefly to the difference in opinion as to the best method of applying the sewage. The original method was to allow the sewage to run upon a bed slowly, perhaps at the rate of 50,000 to 70,000 gallons per acre in six hours. Under such conditions it is difficult to secure even distribution and to avoid the passage of large volumes of sewage through particular areas of the bed. Furrowing helps

somewhat, but it is necessary also to provide wooden carriers in which the sewage may flow to the extreme portions of the bed. Sometimes, as at Brockton, there is a single line of trough running down the center of the bed, narrowing as it proceeds, and discharging a portion of its flow at each decrease in diameter (Fig. 67). In other plants, as at Lake Forest, Ill. (Fig. 68), the troughs radiate out in a crowfoot pattern. At this plant the troughs consist of two upright sides of 2-inch plank, resting on a

FIG. 68. General View of Intermittent Filters at Lake Forest, Ill. (courtesy of J. A. Alvord).

similar bottom plank with 3-inch square holes at the base of the sides, spaced about 2 feet apart, from which the sewage gushes out.

At many of the newer filter beds a dosing tank is provided, from which, by means of automatic devices, the allotted amount of sewage can be run upon the bed in from fifteen to thirty minutes, flooding the bed 6 inches to 1 foot above the surface of the sand. This greatly improves distribution and renders furrowing unnecessary. In the Middle West automatic dosing appa-

ratus has been developed to a considerable degree of perfection. The danger from the failure of such devices is, of course, always considerable, and they absolutely require periodic expert supervision; but even a fairly accurate automatic mechanism may perhaps be considered as reliable as the average city employee. The apparatus installed at Lake Forest by Alvord (Fig. 69) is a simple and ingenious one. A float in the dosing chamber lifts an iron ball in one of a series of wooden columns,

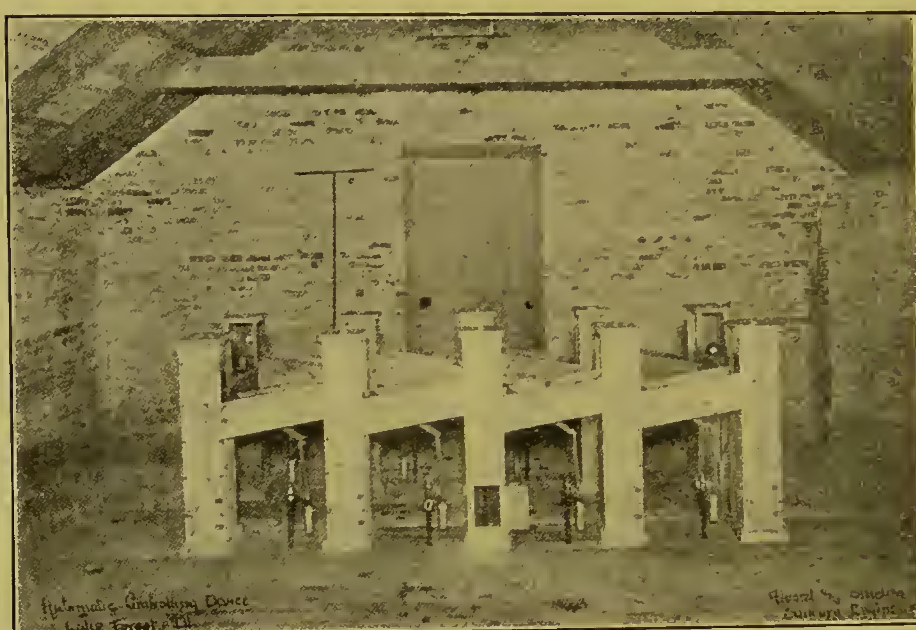


FIG. 69. Dosing Apparatus at Lake Forest, Ill. (courtesy of J. A. Alvord).

and at a certain height the ball rolls through a trough from one column to the next, in its passage striking a catch, which opens an air valve attached to one of ten bell-siphons in the dosing chamber. Each of the siphons discharges on one of the ten sand beds, which are thus dosed in rotation. The general form of siphon used with such devices is shown in Fig. 70.

Operation of Intermittent Filters. The operation of intermittent filtration plants in summer is very simple. Bacterial action is at its best, and all that is necessary is to apply the

allotted dose to each bed, and to break up and remove the deposit that is left on the surface of the sand. The suspended matters of the sewage are retained for the most part near the

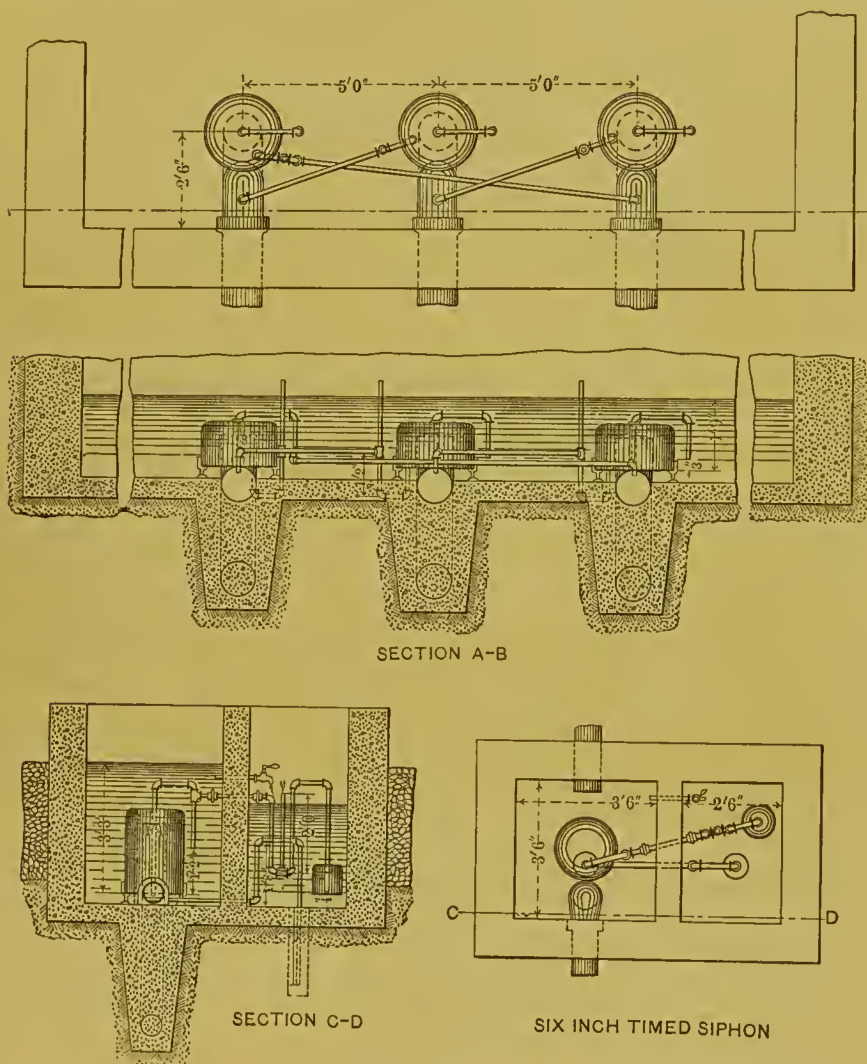


FIG. 70. Miller Automatic Flushing Apparatus for Dosing Intermittent Filters.

surface, and there is danger that they may form an impervious layer and cut off the air supply of the bed. Frequent raking and harrowing, and from time to time removal of the deposit and occasional plowing of the beds is therefore necessary. At

Brockton, Massachusetts, with sewage containing 195 parts suspended matter per million, and receiving practically no preliminary treatment, six to eight tons of solid matter are removed from the beds for each million gallons of sewage filtered. The material is not very offensive and is sufficiently dry to be easily handled. It can be dug into the ground if not disposed of to farmers. Besides this accumulation on the surface of the beds there is a deeper clogging, due to fine particles which penetrate from 3 to 12 inches below the surface of the sand. This lowers the rate of filtration, and may result after a period of years in necessitating the reconstruction of the bed. At Worcester, Mass., owing to the trade waste in the sewage, this deeper clogging was more serious when the sewage was treated with chemicals than when crude sewage was applied. At certain intermittent filtration plants, such as Framingham, Natick and Brockton, crops have at times been grown on the beds, but it has been generally abandoned, except at Framingham.

During the long and cold winters of the northern United States, when for weeks together the temperature does not rise above zero degrees C. and the beds are covered with snow and ice from December to March, so that the surface of the bed cannot be reached, intermittent filters have to be operated with considerable judgment and care to obtain good results. The temperature of the sewage in winter as it comes to a plant is not often below 7 degrees above zero C.; and if a comparatively large amount of this sewage is applied to the bed under the snow and ice, the frost is removed from the sand and the sewage penetrates the bed. There is danger, however, that before all the sewage passes into the bed it may become chilled and a solid layer of ice form on the surface of the sand, which, if it occurs, prevents further sewage being applied to the bed until there has been sufficient warm weather to melt the ice layer. To prevent the freezing up of the bed in this way, the beds are prepared for winter work by furrowing or by making small heaps of sand at frequent intervals on the sand area, since it has been found that, if the sewage does become chilled on a bed so

prepared, the ice that is formed extends from ridge to ridge or from sand heap to sand heap, forming a natural covering over

FIG. 71. Intermittent Filter Bed at Pawtucket, R. I., in winter.

that portion of the area which receives the sewage, and protecting it to a greater or less extent from the action of frost (Fig. 72).

The manner of working the beds is also somewhat different in winter from what it is in summer. Two to four times as much

sewage is applied in a dose as in summer, and the bed is allowed to rest for a proportionally longer time. In this way, as the sewage runs under the snow and ice covering, the frost is removed from the sand and the sewage penetrates the bed. There is a certain amount of danger in this method of procedure, as without competent supervision some of the beds may not be treated with sewage often enough to maintain their nitrifying action.

FIG. 72. Ice on Intermittent Filter Bed in Winter (courtesy of Massachusetts State Board of Health).

There is also trouble in winter, owing to the clogging of the beds and the lack of opportunity for removing the deposit. A mat $\frac{1}{4}$ inch or more in thickness, and resembling papier-maché in appearance, forms on the top of the sand. This prevents the sewage from passing quickly into the sand, and may, if too much sewage is applied, cause a bed to become water-logged. The amount of sewage that can be applied is thus materially reduced, and unless there is a reserve area available for winter use (or, what amounts to the same thing, the area is sufficiently large, so

that the maximum dose applied in summer is much less than that which the area would purify), there is danger of a certain amount of sewage being allowed to escape untreated in winter.

Under the best conditions the effluent from a sand filter is not quite so good in winter as in summer. The nitrifying organisms are very sensitive to decreased temperature, even a difference of two or three degrees showing its effect in their diminished activity. Winslow and Phelps (1906) prepared the table below to show the monthly variations recorded for the experimental filters at Lawrence and for the filtration area at Brockton.

TABLE LI. — MONTHLY VARIATIONS IN SAND-FILTER EFFLUENTS AT BROCKTON AND LAWRENCE, MASS.

(Yearly average = 100.)

	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec
FREE AMMONIA.												
Brockton....	100	163	192	171	134	92	79	63	50	50	50	83
Lawrence....	213	269	204	168	84	48	16	6	8	14	32	120
ALBUMINOID AMMONIA.												
Brockton....	87	130	130	174	87	87	87	87	87	87	87	87
Lawrence....	175	171	150	132	92	76	67	58	62	49	62	117
NITRATES.												
Brockton....	85	70	67	81	90	112	118	115	133	129	110	96
Lawrence....	48	38	59	100	139	140	120	116	128	125	109	75
OXYGEN CONSUMED.												
Brockton....	84	132	152	178	100	84	72	68	68	84	92	108
Lawrence....	170	167	149	122	84	70	69	61	62	64	61	124

The Lawrence figures are the averages of the ratios for tanks Nos. 1, 2, 3, 4, 6 and 10, from 1895 to 1900, calculated from Clark's analyses. For Brockton the figures used cover the period

from 1897 to 1904. A regular seasonal variation is indicated, the organic constituents reaching their maximum in February with the small Lawrence tanks, and in March with the large Brockton filter. The nitrates show a reciprocal curve, lowest in February. The maximum monthly deviation amounts to about 100 per cent, the worst monthly averages being twice as high as the general average in organic matter.

Nevertheless, even in winter the effluent from a well-managed filtration area is generally stable, when submitted to incubation tests.

The rates at which intermittent filters can be operated vary from 20,000 to 200,000 gallons per acre per day, the coarseness of the filtering material and the strength of the sewage being the chief controlling factors. As a matter of practical experience, one to two acres of filter surface are allowed for each thousand persons. This corresponds roughly to a rate of 50,000 to 100,000 gallons per acre per day, but the lower rate is nearer to the average.

The Brockton Filter Plant. One of the best examples of intermittent filtration in the United States is the plant at Brockton, Mass. It was built in 1894 by F. Herbert Snow, now Chief Engineer of the Pennsylvania Department of Health, and at that time City Engineer of Brockton. This plant has been operated with exceptional care and skill by C. R. Felton, City Engineer of Brockton, and by G. E. Bolling, the chemist in charge. The following description of this plant taken from Goodnough and Johnson's account of sewage disposal in Massachusetts (1899), and from the Annual Reports of the City Engineer and the Sewerage Commissioners of the City of Brockton and supplemented by much valuable information received from Mr. Bolling, may be taken as a type of the most approved practice.

Brockton is a city of about 53,000 inhabitants (27,294 in 1890, 40,063 in 1900, 47,794 in 1905), situated in a level sandy region of eastern Massachusetts, about seventeen miles from the ocean. The city has had a water supply, from surface sources,

since 1880. A sewerage system was first put in operation in 1894, and the problem of disposal had to be met at once, as the only stream near by, the Salisbury Plain River, is too small to receive any considerable body of untreated sewage. Provision was therefore at once made for conveying the sewage by main sewers to a pumping station located in the southerly part of the city. From this point it was planned to pump it to a filtration area located about three miles and a half away, upon land draining into the headwaters of the Taunton River.

The water consumption in Brockton is low, 35.5 gallons per capita, and as the city is sewered on a separate system and great care has been taken to exclude ground water as far as possible, the sewage, as shown by the analysis given in Chapter I, page 6, is a strong one.

The sewage as it arrives at the pumping station is received in a covered masonry reservoir with a capacity of 619,000 gallons, designed to hold the night flow, in order that the pumping may be done in the daytime. From this reservoir the sewage passes through screens consisting of iron slats with an open space between them of three-quarters of an inch, and thence passes to the pumps.

The screens are so arranged that they are entirely submerged when the reservoir is full, and the entire screen area of about 100 square feet is available at that time. As the level of the sewage in the reservoir goes down, the available screen area is diminished, the bottom of the screens being but slightly below the bottom of the reservoir. The screens are cleaned several times each day, while the pumps are being operated, and the amount of the screenings averaged 499.1 pounds per day in 1907. The material thus removed is burned beneath the boilers.

The solid matter which accumulates at the bottom of the reservoir during the night is removed by stirring it up when there remains only about a foot of sewage in the reservoir, and when pumped to the filtration area is delivered on special "sludge beds." The stirring is effected by means of an agitator, which consists of perforated pipes laid on the bottom of the reservoir

and connected with the force main, through which sewage can be discharged under a head.

The sewage is forced from the pumping station to the filter beds by two pumping engines, used on alternate days, each having a capacity of 5,000,000 gallons in twenty-four hours. These pumps are provided with ordinary water valves, which are said to require frequent attention, on account of substances getting caught beneath them.

The force main from the pumping station to the filtration area is a cast-iron pipe 24 inches in diameter and 17,500 feet in length. The force main remaining full from one day to another, gives a combined storage of 1,030,000 gallons in both reservoir and force main. In addition to this a dâm has been constructed in the main intercepting sewer, giving an additional storage of 175,000 gallons, so that altogether 1,205,000 gallons can be stored, or more than the average daily pumpage to the beds, which was 1,164,000 gallons in 1907. Since the force main always remains full, however, the net storage capacity is but 794,000 gallons, or about 69 per cent of the average daily pumpage in 1907. Since the pumping is carried on during the hours of heaviest flow, a large amount of comparatively fresh sewage is pumped to the beds, as well as that which has undergone considerable septic action during storage.

The filtration area is located in the southwesterly corner of the city, adjoining the towns of Easton and West Bridgewater. The original plant comprised 23 beds; seven new ones were added in 1905, and seven more in 1908. The general plan of the plant is shown in Fig. 73; and the provision of a laboratory building where the operation of the beds is controlled by frequent and regular analyses may be noted as a most important feature of the plant. At small plants, to reduce expense, such a laboratory can also be developed, as at Brockton, into a general municipal laboratory.

The average area of the beds at Brockton is about one acre each; but shape and size are varied to suit the configuration of the ground. A large peat hole rendered about two acres of the

FIG. 73. Plan of Brockton Filter Beds (courtesy of G. E. Bolling).

area unfit for use. The beds were prepared for receiving sewage by the removal of all of the loam from the surface. The sand is stratified, but the different strata are not separated, in most cases, by a distinct line of stratification, and the material as a rule is coarse and porous. In general, the underdrains are laid about 60 feet apart and at a depth of 7 to 9 feet. They discharge into main drains, varying in diameter from 8 to 15 inches, which carry the effluent into the Coweeset River, or into a small tributary of the river.

The sewage is distributed on the surface of the beds by carriers laid across the bed from the center of one side. They are simply flat-bottomed sluiceways reducing from a width of 5 feet at the inlet to 1 foot at the extreme end by a series of offsets of 6 inches on each side. At each offset is an opening controlled by slanting wooden gates, by which the amount of sewage discharged on any portion of the bed may be controlled. As originally constructed the entire carrier was made of wood, but these have been replaced by carriers with concrete bottoms 5 inches deep, and having the angle irons to which the wooden sides are fastened embedded in the concrete. The initial cost of such carriers is less than when made entirely of wood and they are much more permanent. A view of the bed with its carriers is shown in Fig. 67.

The general method of operating the beds is as follows: The solid matter which accumulates in the bottom of the reservoir is mixed with a small volume of sewage and pumped into the force main at the end of the day's run. In the lower part of the force main it remains over night and is discharged at the filtration area with the first morning pumping. This heavy material is run on 5 special beds called "sludge beds," the amount applied to each bed being about 100,000 gallons once in four days.

The five so-called "sludge beds" have to be cleaned practically once a month and the intermittent filtration beds at least three times a year. The total amount of deposit removed from the beds in 1907 was 3422 tons, or approximately 8 tons per million gallons. Often the dry deposit can be removed in large

sheets with hay forks. When shoveling is required, more or less sand is necessarily carried off with the deposit.

After the very heavy sewage, which averages 120,000 gallons per day, has been delivered upon the special beds, the lighter sewage is discharged upon the other beds, each bed receiving sewage for 25–30 minutes, the dose applied at one time being from 115,000–140,000 gallons. Each bed is then rested for three or more days before the next dose is applied. The average rate per acre for the whole plant is about 30,000 gallons per day.

During the first years the rakings were burned on woodpiles in the open fields. The odor of the fumes was so obnoxious that, in 1898, the burning was discontinued, and the rakings were disposed of to farmers at a small price, the amount received from 1900 to 1906 for the total rakings being about $150 a year. In 1906, in order to have the rakings removed promptly, no charge was made, and in 1909 the city was fortunate enough to make a contract for five years for the disposal of all the rakings, the contractor to remove these rakings immediately, free of cost to the city, whenever a bed was cleaned. The composition of the rakings varies with the season, the moisture ranging from 5 to 50 per cent.

An average of several analyses gave the following result:

Moisture	16.22
Phosphoric acid	.78
Potassium oxide	.51
Nitrogen	1.45
Calcium oxide	.30
Insoluble matter, sand, etc.	70.13

In spite of the care with which these beds have been operated there has been a progressive clogging of the upper layers of the sand by the penetration of fine solid material. This manifested itself in the diminished rate at which the sewage would pass through the beds. In 1905–06 about 6 inches of sand was removed from the surface of all the old beds, with marked improvement in the results. At one time crops were raised on some of the beds during the summer, corn being the principal

product, but it was found that cropping was injuring the beds by forming an accumulation of clogging humus material in the upper layers of the sand, and this was abandoned in 1902.

The beds are prepared for winter work by plowing, the surface being left in ridges and furrows, so that an ice sheet may form over the ridges, leaving the furrows as protected covered channels. During the months of December, January, and February little or nothing can be done to the surface of the beds, and at this time the greatest strain is put on the capacity of the plant. It operates with reasonable success, however, at all seasons, and yields an effluent which is well purified and nonputrescible.

The average analytical results for the sewage and six effluents for the year 1907 are shown in the table below. The effluents are of course at their best in the late summer and at their worst in the early spring. The clogging of the beds hinders aeration, so that at this season the effluents frequently contain no dissolved oxygen. The general relation between the purification effected at different seasons is indicated in the table on page 245.

TABLE LII

CHEMICAL COMPOSITION OF SEWAGE AND EFFLUENTS AT BROCKTON

Parts per million. (Brockton, 1908.) Average for year, 1907.

	Free ammonia.	Albuminoid ammonia.	Chlorin.	Nitrogen as — Nitrates.	Nitrogen as — Nitrites.	Oxygen consumed.
Sewage	59.7	3.00	137.4	....	....	416.7
Effluent A	20.5	.85	111.5	1.18	.019	15.7
B	17.2	.72	116.8	5.02	.147	14.6
C	23.2	.91	122.4	.62	.000	15.0
D	10.1	.31	115.0	12.37	.093	4.8
E	18.4	.51	115.0	5.84	.086	8.2
F	16.9	.47	112.5	6.44	.157	8.6

The per cent reduction, as measured by free ammonia, ranges from 60 per cent for effluent C to 83 per cent for effluent D. The purification as judged by the albuminoid ammonia and oxygen consumed averages over 96 per cent for all the effluents. Perhaps the best testimony to the efficiency of the plant is fur-

nished by the analyses of the Coweeset River, above and below the entrance of the effluent.

TABLE LIII

EFFECT ON COWEESET RIVER OF BROCKTON SEWAGE EFFLUENT

Parts per million. (Brockton, 1908.) Average for 1907.

Coweeset River.	Free ammonia.	Albuminoid Ammonia.	Chlorin.	Nitrogen as — Nitrates.	Nitrogen as — Nitrites.	Oxygen consumed.	Dissolved oxygen. Per cent.
Above......	.062	.292	7.7	.191	,001	9.7	79
Below......	2.730	.318	35.2	1.394	.022	8.8	72

Bacterial examinations are not regularly made at the Brockton plant, but a series of tests, made by one of the writers on four different days during the autumn of 1908, gave the following results:

TABLE LIV

BACTERIA IN SEWAGE AND EFFLUENTS AT BROCKTON

Average of four examinations, autumn of 1908.

	Bacteria per cc. Gelatin, 20°.	B. coli per cc. Lactose Bile.
Sewage........................	3,150,000	150,000
Effluent A..................	1,900	400
B..................	6,300	15
D..................	125	0
E..................	1,400	5
F..................	2,000	1

The cost of the first twenty-three beds at Brockton was \$9234 for land and \$50,301.97 for construction. The second set of seven beds cost \$24,438.92 for construction; the third set of seven beds cost \$23,239.06 for construction. The land for the last two sets cost \$10,510; but only a small part of this is in use. The total first cost of the whole plant, including the laboratory may be put at \$122,665.95. Up to and including the year 1907, the capital cost was \$99,426.89. The interest on this sum at 3½ per cent would amount to a capital charge of \$3480; or \$8 per million

gallons of sewage treated in 1907. In 1907 the cost of labor at the disposal area was $5745.22, of which $4355.03 was for the care of the surface of the beds. In addition, the laboratory and other minor items bring the total operating cost for that year to $6805.22. On the basis of 425,000,000 gallons of sewage treated this amounts to $16 per million gallons; or, plus interest charges on the capital cost of the plant, to $24 per million gallons.

Combination of Intermittent Filtration with Preliminary Processes for the Removal of Suspended Solids. The older intermittent filtration areas in Massachusetts, almost without exception, treat crude or only very roughly sedimented sewage. At Brockton, as has been pointed out, the heavier portion of the day's pumping goes on special "sludge beds"; but everything passes to some part of the filtration area. Crude sewage can be treated by this process with good results. The area required for a given community is, however, chiefly determined by the amount of sewage which can be passed during the winter months without scraping the beds; and this quantity is fixed by the amount of suspended matter in the sewage.

A dose of 100,000 gallons on an acre corresponds to a depth of less than four inches of sewage. With a bed of fairly coarse sand, well-leveled and equipped with good distributors, a dose of well-clarified sewage disappears in half an hour and may be repeated as often as once every six hours without interfering at all with nitrification, as indicated by recent experiments at the Technology experiment station. The deposit must, however, be removed from such a bed four times as often as from one operated at the ordinary rate; and it is impossible to do this during freezing weather. Where unlimited areas of sand *in situ* are not available, it is worth while to subject the sewage to preliminary processes for the removal of solids and to operate the filters in a more intensive fashion. Careful preparation of the beds, frequent and regular dosing and skilled supervision are required; but by these means the rate of filtration may perhaps be raised from 50,000 gallons per acre per day to 200,000.

One of the largest and best constructed plants in which the sewage is subjected to preliminary treatment for the removal of suspended solids is at Saratoga, N. Y. This plant has been mentioned in Chapter VI, where a full description of the septic tanks and the purification they effect is given. The intermittent filtration beds, twenty in number, — eighteen of them about one acre in area, the others somewhat smaller, — consist of sand with an effective size of 0.20 mm. and a uniformity coefficient about 2. This sand extends to an unknown depth and the ground water level is about 16 feet below the surface. There is only one line of under drains in each bed, at a depth of 6.5 feet, and a line of 10–15 inch drain, at a depth of 11 feet, with which the smaller drains might connect. In the main drain manholes were placed at the junction of the laterals, and at the end of all drains, which were turned up and carried above the surface of the bed, air vents were provided. In this way circulation of air can take place, it is believed, with beneficial effect in the reduced accumulation of carbonic acid gas in the body of the filters. On account of the depth of the sand only about a quarter of the liquid discharged upon the beds finds an outlet through the drains, the greater part running down through the sand without appreciably raising the water table. In midsummer twelve beds are used daily, the gates being changed twice; during the remainder of the year eight beds are used each day, one shift of the gates being necessary.

The average daily amount of sewage per bed in use is about 140,000 United States gallons, applied in four doses. The entire field is kept in commission and the beds used alternately, so that the average rate per day for the field is about 60,000 gallons per acre. Mr. Barbour, who constructed the plant, believes that double this rate could be maintained with equally good results.

In the central regions of the United States, sand of suitable quality for filter beds must often be transported from considerable distances. The intensive operation of the sand filter has therefore reached a high degree of development in these states, particularly in Ohio and in Wisconsin, the removal of solids from

the sewage being generally accomplished by preliminary treatment in the septic tank. In Ohio, for example, there are now 26 intermittent filtration areas, 12 of which are dosed with sewage from which the solids have been more or less carefully removed.

Two good examples of the combination septic-intermittent-filter plant are in operation in the town of Wauwatosa, Wis. The first of these is a small plant which serves the town itself and was built about 1901 by Alvord and Shields. It was visited by one of the authors (Winslow, 1905 *b*), during the winter of 1904–05, when some 200 connections had been made with the sewer system, yielding a flow of about 100,000 gallons per day. The septic tank, of concrete, sheltered by a brick roof, was approximately 15 feet by 50 feet by 10 feet deep, with a capacity of 40,000 gallons, giving a storage period of 10 hours. According to the town engineer, the tank was cleaned out twice a year, a quantity of combined scum and sludge equal to half its capacity being removed by dipping out with pails and by the use of a small rotary pump. The effluent appeared like a good septic sewage, dark gray in color and with no large particles.

The septic effluent passes to a dosing chamber in a separate small brick structure, to be discharged on sand beds by an automatic device. The beds are 6 in number, 30 feet by 60 feet, with a combined area of one-fourth acre, thus giving a rate of 400,000 gallons. The sand used is coarse and the results obtained are said to be excellent. In cold weather, however, it is the practice of the authorities to discharge the septic effluent directly into Menominee Creek without filtration.

In the same town is a larger plant of almost exactly similar construction (Shields, 1904), which shows what good results can be obtained by careful and efficient operation. The Wauwatosa County Institutions form a group of five buildings, including two insane hospitals, an almshouse, a county hospital and a home for dependent children. The total population is about 3500, and the water consumption, 400,000 gallons per day. A chemical precipitation system was put in in 1889, the dosing house and coagulating basin still remaining as its monument. Then a sep-

tic tank was substituted; but it proved unsatisfactory, and in 1904 Mr. Shields built a new septic tank and filter beds. The new tank is 85 feet long, 20 feet wide and 8 feet deep, with a central longitudinal partition and 3 concrete baffles. The storage period, under the conditions existing in 1905, appeared to be 6 hours. With the addition of the old septic tank, 8 feet by 17 feet by 54 feet, it would be increased to 9 hours. Both tanks were of concrete, housed under low brick buildings with wooden roofs.

From the septic tank the sewage flowed through an inverted siphon to the filter beds, located on the farther side of a small stream. The beds were eight in number, arranged in two rows, with a controlling house in the center. The four corner beds were each 50 feet by 110 feet, while the four center beds were 57.3 feet by 55 feet, being shortened to provide room for the distribution system. The latter was of the general pattern described on page 240, including a dosing chamber discharged by any one of eight 15-inch siphons, each connected with one bed. The rotation of the beds was controlled by an automatic device.

The filter beds were built up of 12 inches of coarse gravel, 12 inches of fine gravel and 12 inches of coarse sand, and were underdrained by four lines of 4-inch pipe. The carriers were of the usual type — two straight troughs in each bed with 3-inch square holes about 2 feet apart.

The plant, when seen in 1905, was carefully supervised by the superintendent of the institution, and was working in admirable shape. The siphons flushed perhaps once every 35 minutes in the morning, every 45 minutes in the afternoon, and once an hour at night, so that each bed was dosed once in from 4 to 8 hours. The total area was about 1 acre for the 400,000 gallons treated.

In spite of severe weather in January, the dose disappeared in twenty minutes after its application; but at intervals it was necessary to rest a bed for a few days by putting into the regulator a chute to shut out one of the dosing siphons. The effluent from the plant, as seen flowing into the Menominee Creek, ap-

peared clear and well purified (Winslow, 1905 *b*). No analyses are obtainable from either of these plants.

It must be clearly understood that the object of the preliminary septic treatment in such plants is solely the removal of suspended solids so as to diminish clogging. It has been claimed in England by Martin, Cameron and Fowler that the soluble elements in a septic effluent are more easily nitrified than those in fresh sewage. There is little evidence in favor of this view. On the contrary, there is clear evidence that the septic process may be carried so far that the effluent is less easily nitrified than it would be without such treatment. At Andover, Mass., for example, where the sewage is strong, and already twenty-four hours old when it reaches the disposal area, the introduction of a septic tank proved distinctly harmful. It seems probable that any trouble of this kind can easily be overcome by aeration between the septic tank and the sand filters. At Saratoga Springs, N. Y., the septic effluent is made to flow in a thin sheet over circular perforated sheet-iron plates sixteen feet in diameter, with the result that the oxygen saturation rises to 70 per cent. It is interesting to note, as indicating the avidity with which such a liquid absorbs oxygen, that the saturation value falls to 40 per cent by the time the effluent reaches the filters (Barbour, 1905).

Practical Results of Intermittent Filtration. The results of intermittent filtration, where the process has been carefully supervised and the beds have not been overdosed, have been excellent, but there are many examples in Massachusetts where the reverse is true. The beds at Gardner have had to be entirely rebuilt, and serious trouble has been experienced at Westboro, Andover and Marlboro. This is not the fault of the method, but usually of an attempt to purify too large an amount of sewage on a given area, and lack of skilled supervision. The method of intermittent filtration is not "fool proof," and the best constructed plant can easily be so mismanaged as to become practically a total failure.

At the best plants the purification effected varies consider-

ably at different seasons of the year. The worst effluent is produced during the spring, when the surfaces of the beds are clogged, but as this is the time of year when there is the largest flow in the streams, the imperfectly purified effluent has less

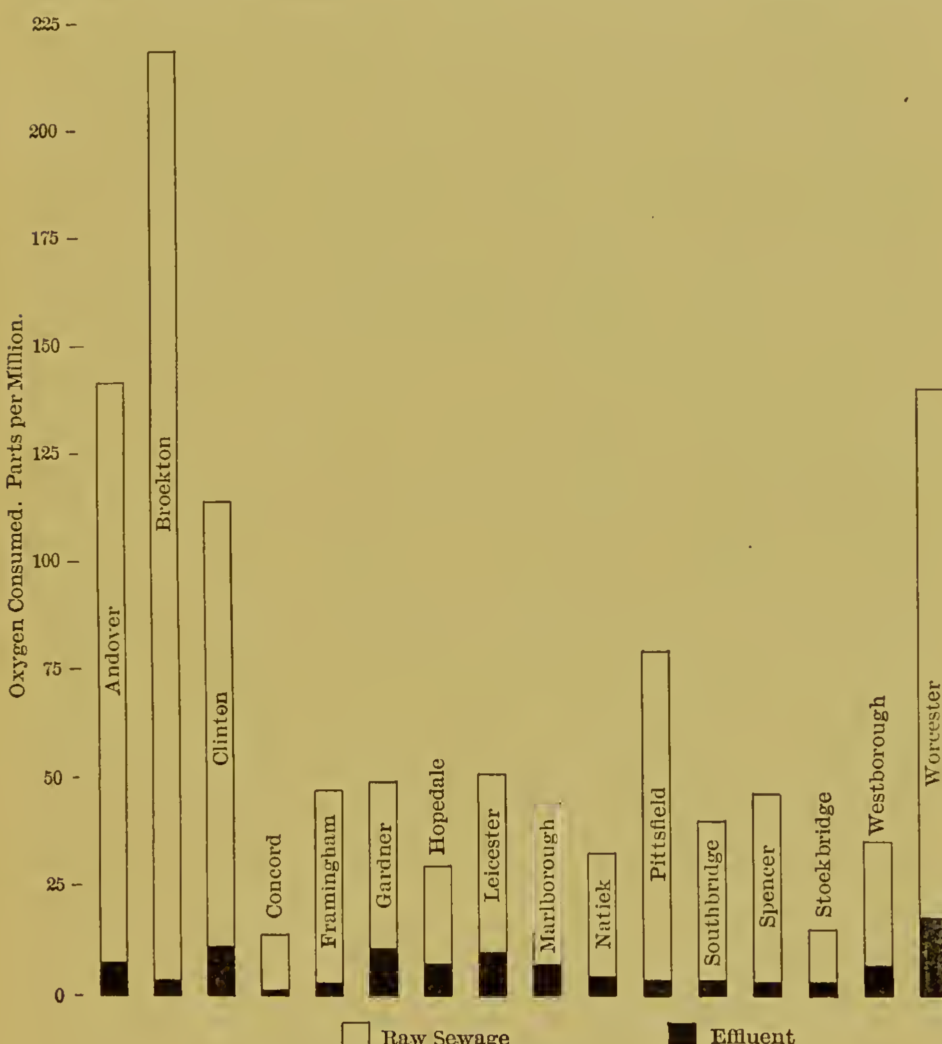


FIG. 74. Diagram of Purification Effected by Intermittent Filters.

effect than it would at other seasons. Experience has shown that if sewage is all passed through sand filters, having a depth of at least four feet, the effluent will be nonputrescible, and consequently cause no trouble when discharged into flowing water.

TABLE LV

STATISTICS OF INTERMITTENT FILTRATION IN MASSACHUSETTS

(Massachusetts, 1904; Worcester, 1905.)

Place.	Date of construction.	Average daily flow (million gallons).	Area of filter beds (acres).	Rate (million gallons per acre per day).	Population contributing sewage.	Preliminary treatment.	Material.	Average of monthly analyses, 1903 (parts per million). Nitrogen as — Free ammonia.	Albuminoid ammonia.	Nitrates.	Nitrites.	Oxygen consumed in 2 minutes, boiling.
Andover........	1898	0.125	3.65	0.0342	3,600	Screening; sedimentation.	Sewage	39.7	5.6	......	......	49.0
							Effluent	9.1	.5	8.3	.2	7.4
Brockton.......	1894	.878	21.48	.0409	25,000	Partial sedimentation .	Sewage	43.8	12.9	......	......	219
							Effluent	1.9	.1	30.8	.1	3.3
Clinton.........	1899	.785	23.5	.0334	10,000	....do..............	Sewage	33.2	7.9	......	......	113.7
							Effluent	.8	.6	4.4	.2	11.2
Concord.......	1898	.312	3.3	.0945	1,200	Screening...........	Sewage	5.7	1.4	......	......	13.6
							Effluent	.01	.09	8.5	0	1.1
Framingham....	1889	.652	19.9	.0328	7,500	....do..............	Sewage	26.1	6.5	......	......	47.3
							Effluent	1.8	.2	9.9	.2	2.6
Gardner (Gardner system)...	1891	.302	2.5	.1208	3,500	Sedimentation........	Sewage	20.2	4.9	......	......	49.2
							Effluent	14.8	.7	.4	0	10.7
Hopedale.......	1892	.150	2.3	.0652	2,000	Septic tank..........	Sewage	18.3	2.8	......	......	29.8
							Effluent	8.6	.7	15.9	.3	7.2
Leicester.......	1894	.030	.36	.0833	500	Sedimentation.......	Sewage	22	4.1	......	......	50.8
							Effluent	5.8	.7	9.1	.4	9.6

TABLE LV (*Continued*)

STATISTICS OF INTERMITTENT FILTRATION IN MASSACHUSETTS

(Massachusetts, 1904; Worcester, 1905.)

Place.	Date of construction.	Average daily flow (million gallons).	Area of filter beds (acres).	Rate (million gallons per acre per day)	Population contributing sewage.	Preliminary treatment.	Material.	Average of monthly analyses, 1903 * (parts per million). Nitrogen as — Free ammonia.	Albuminoid ammonia.	Nitrates.	Nitrites.	Oxygen consumed in 2 minutes, boiling.
Marlboro.......	1890	1.100	11.2	.0982	10,000	Sedimentation	Sewage Effluent	25.9 10.2	4.4 .5	...... 3.5	3	44.4 7.3
Natick.........	1895	.566	11.1	.0510	4,000	Screening...........	Sewage Effluent	12.2 5.1	2.6 .3	...... 2.2	2	32.7 4.4
Pittsfield.......	1890	1.456	21.67	.0672	15,000	....do..............	Sewage Effluent	12.0 2.2	8.2 .3	...... 6.7	2	79.2 3.8
Southbridge....	1899	.350	7.25	.0483	2,200	....do..........	Sewage Effluent	16.0 3.2	3.5 .3	...... 2.0	1	40.1 3.6
Spencer........	1897	.375	9.3	.0403	3,000	....do..............	Sewage Effluent	1.5 1.2	4.5 .2	...... 3.7	2	46.5 3.3
Stockbridge.....	1899	.075	3.6	.0208	800	....do..............	Sewage Effluent	9.8 0.9	1.6 .2	...... 1.4	0	15.2 3.3
Westboro.......	1891	.282	4.0	.0705	3,000	Screening, partial sedimentation.	Sewage Effluent	13.8 5.3	4.4 .7	...... 3.6	5	35.5 7.3
Worcester......	1898	1.080	8.77	.123	122,000	Sedimentation........	Sewage Effluent	18.0 10.9	8.1 .9	.6 2.0	.1 .3	138.7 18.1

* Worcester, 1904.

If, on the other hand, through overdosing or mismanagement the beds have become clogged and the sewage passes merely over and not through them, objectionable conditions must arise.

A fair idea of the general practice of intermittent filtration in Massachusetts and of the amount of purification obtained may be gained from Table LV, compiled from the Thirty-fifth Annual Report of the Massachusetts State Board of Health; and the data for "oxygen consumed" are plotted in Fig. 74 (Winslow and Phelps, 1906).

With regard to the comparative results of different Massachusetts plants the following points may be noted: The poor effluents at Westboro and Gardner are in part due to careless operation, the sewage being allowed to run on continuously for days. At Clinton the applied sewage is very strong. At Leicester, Andover and Hopedale the Board attributes results below the average to the fact that the sewage is stale or septic when applied. At Worcester the sewage is strong, and it is probable that acid-iron waste interferes somewhat with the process of nitrification.

TABLE LVI

BACTERIA IN SEWAGE SEPTIC EFFLUENT AND SAND-FILTER EFFLUENT AT IOWA STATE COLLEGE

(Walker, 1901.)

Month.	Bacteria per c.c. Monthly averages.		
	Sewage.	Septic effluent.	Sand effluent.
August, 1899	2,392,600	1,388,300	2,246
September	8,815,000	3,245,000	3,660
October	6,064,800	4,941,000	4,320
November	4,537,333	3,014,000	2,261
December	816,333	848,000	2,319
January, 1900	848,000	726,000	830
February	345,533	233,810	3,451
March	132,125	112,500	2,480
April	2,121,000	1,392,800	13,263
May	1,021,000	783,300	3,077
June	1,318,100	1,391,300	2,359
July	3,908,700	4,578,333	2,270
August	403,118	215,700	546
September	1,181,533	383,733	850

The bacterial purification effected by an intermittent filter should be over 99 per cent if the plant is carefully operated. Bacterial results have been already quoted for Brockton, and the table above shows the purification obtained at Ames, Iowa. When filters are badly constructed or overdosed, both chemical and bacterial results will probably fall below the values quoted.

The Cost of Intermittent Filtration. Intermittent filtration, like all other processes of sewage purification, cannot be carried out except at considerable expense. The first cost of beds built under most favorable conditions and where sand is used *in situ*, may be as low as $1000 or $1500 per acre. Where the sand cannot be used *in situ*, or in other words, where the beds are artificially constructed, the cost will be materially increased. $3000 per acre is perhaps a fair average cost under all conditions, but it not infrequently runs up to $5000 or more.

Intermittent filter beds are actually operated in Massachusetts at rates of between 50,000 and 100,000 gallons per acre per day. By including a tank in which a large proportion of the suspended matter is removed, they may be operated at higher rates, as already stated.

Assuming $1250 as the cost per acre of filters built under the best conditions, and 75,000 gallons per acre per day as the rate of filtration, the annual interest charges, at $3\frac{1}{2}$ per cent upon the capital expenditure, will amount to $1.60 per million gallons. Assuming an average cost of $3000 per acre and the same rate of filtration, the annual interest charges, at $3\frac{1}{2}$ per cent, will amount to $3.84. While these rates indicate the interest charges which should be anticipated for such plants, built under the best and under average conditions respectively, these charges may, under less favorable conditions, run as high as $6.50, or even higher. At Brockton, as noted above, the actual cost has been about $8.

The cost of operation of intermittent filtration plants is somewhat larger than the interest charges, because the surface of the beds demands a large amount of care to maintain a condition of reasonable efficiency. The operation costs at Brockton have

already been discussed. They are undoubtedly high on a million gallon basis, on account of the strength of the sewage; but on a per capita basis, they correspond closely with Mr. Fuller's estimate (Fuller, 1909) of 20 cents per capita per annum. The expense of maintenance at 16 Massachusetts plants varied in 1903 from \$0.61 per million gallons at Natick to \$21.92 at Stockbridge (Massachusetts, 1904). The Natick beds received practically no care except in connection with cropping, but operated satisfactorily with a relatively small amount of sewage. The next lowest figures were \$2.45 at Pittsfield and \$2.60 at Marlboro (large plants) and \$2.87 at Concord (weak sewage). The costs at Clinton, Southbridge, Spencer, Westboro and Worcester were between \$3.91 and \$7.80; at Gardner, over \$9 at each of its two areas; at Leicester, \$11; and at Andover, \$13.98. It may be considered, perhaps, that \$7.50 per million gallons is somewhere near a fair figure for the average American sewage, rising perhaps to \$10 or \$15 with a strong sewage like that of Brockton.

A total cost of ten to twenty dollars per million gallons for operating expense and interest charges is not exorbitant when the excellent character of the effluent from intermittent filtration is taken into account. When a high degree of purification is required, and when sand for filtration is available, this method unquestionably furnishes an ideal solution of the problem. If a less perfectly purified effluent will serve, other processes of treatment may prove more economical. The availability of intermittent filtration is, in any case, directly dependent upon the proximity of suitable sand areas. In regions where there are no deposits of good sand within reach the cost of transportation and construction would generally prove prohibitive.

CHAPTER X

PURIFICATION OF SEWAGE IN CONTACT BEDS

Historical Development of the Contact Bed. The general method of sewage disposal worked out at Lawrence had only a limited application, being quite unsuitable for regions not provided with ample deposits of sandy soil. For many communities the cost of constructing sand filters of sufficient area to treat sewage, at a rate of 100,000 gallons per acre per day, would be entirely prohibitive. In England, where sand is not of common occurrence, it was absolutely necessary to modify the process so as to obtain higher rates of filtration. It was in England, therefore, that the newer methods of sewage purification, the so-called "biological processes" were developed, based like intermittent filtration on the oxidizing activity of micro-organisms, but scientifically controlled and regulated so as to be more intensive in their action.

W. J. Dibdin, Chemist to the London County Council, was one of the first English sanitarians to grasp the essential principles of sewage purification. In studies of the self-purification of the Thames, H. C. Sorby had pointed out, as early as 1883, the part played by living organisms, although he had in view chiefly the consumption of solids by the larger microscopic forms. In 1884 Dupré went a step further in affirming the relation of organic life to the oxidations which take place in a purifying stream. Dibdin, who had been associated with both these observers, read a paper before the Institution of Civil Engineers in 1887, in which he worked out the whole theory as follows:

"In all probability the true way of purifying sewage, where suitable land is unavailable, will be first to separate the sludge, and then to turn into the effluent a charge of the proper organism, whatever that may be, specially cultivated for the purpose, and retain it for a sufficient period, during which time it should be

fully aerated and finally discharged into the stream in a really purified condition. This, indeed, is only what is aimed at and imperfectly accomplished on a sewage farm."

The treatment of London sewage by chemical precipitation had been recognized by the Metropolitan Sewage Commission of 1884 as only a temporary expedient, purification by land filtration being contemplated as the ultimate outcome of the difficulty. As soon as the Massachusetts results were published, Dibdin saw that they gave promise of a better solution of the London problem than could be found in the laying out of immense sewage farms. Intermittent filtration did not, however, suit the case. The problem was to purify sewage on still smaller areas than those used in the Lawrence experiments. Obviously, in order to accomplish this end it was necessary to build filters of material coarser than sand or gravel. With coarse material, however, frictional resistance could no longer be depended on to delay the passage of sewage through the bed and give time for the purifying agencies to work. It was necessary, therefore, to regulate the flow by constructing water-tight filters in which the sewage could somehow be retained in "contact" with the filling material and its accumulated growth of micro-organisms for the requisite period of time.

Dibdin's first experiments on the purification of sewage at high rates were carried out at one of the London sewage outfalls between May and August, 1892. Four wooden tanks were installed at the northern (Barking) outfall. Each was 5 feet deep and had an area of one two-hundredth of an acre. The tanks were filled, respectively, with burnt clay, pea ballast (Lowestoft shingle), coke breeze, and a combination of gravel and sand over a layer of proprietary material. All received effluent from the chemical precipitation tanks at an average rate of 400,000 gallons per acre per day. Sewage was allowed to run through continuously for eight hours, the rate being controlled by partially closing the outlet valves, and the beds were allowed to stand empty for aeration during the remainder of the twenty-four hours. All four filters yielded effluents which were purified to a

very considerable extent, the oxygen consumed and albuminoid nitrogen values being reduced to a half or a third of the amount in the applied liquid. Of the four filling materials, the coke breeze proved most satisfactory, the coarser burnt clay yielding a much poorer effluent, and the sand clogging seriously and giving a clear but imperfectly purified filtrate.

Coke breeze was therefore fixed upon as the best material for further experiments, and a second series was begun to study the details of practical operation on a larger scale. A filter bed 1 acre in area, consisting of 3 feet of pan breeze covered with 3 inches of gravel, was constructed at Barking and put into operation in September, 1893. At first the bed was dosed too heavily and soon became clogged and foul. The need of rest and aeration, especially when a new filter is first operated, was thus clearly shown. After three months' rest the bed could handle two fillings a day, the sewage being allowed to stand in it for a period of from one to two hours. The cycle finally established allowed one and one-half hours for filling the bed, two hours for standing full, two and one-half hours for emptying, and six hours for aeration. When gradually worked up to its full capacity sewage could be treated at a rate of 1,200,000 gallons per acre per day. Though the purification effected was not at all equal to that obtained by intermittent filtration, the results, as shown by the following table, were surprisingly good.

TABLE LVII

RESULTS OF CONTACT TREATMENT AT LONDON, ACRE FILTER

Parts per million. (Clowes and Houston, 1904.)

Period.	Oxygen consumed in 4 hours at 80° F.		Nitrogen as —			
			Free ammonia.		Nitrates.	
	Chemical effluent.	Filter effluent.	Chemical effluent.	Filter effluent.	Chemical effluent.	Filter effluent.
September–December, 1893...	59	17	4.8	1.4	1.6	1.9
April–June, 1894............	59	12	4.9	1.1	1.8	3.4
July–November, 1894........	52	10	4.7	1.3	.3	2.0
January–March, 1895........	61	14	4.8	1.4	5.4	9.7
April–September, 1895.......	46	9	4.0	1.3	2.0	7.6
May–June, 1897.............	31	6	3.0	.9	.6	4.1
1900–1901..................	55	9	......	......	.4	10.0

The early London experiments of Dibdin have been greatly extended since 1898 by Clowes and Houston. Various details of construction and operation were worked out at both the Barking and Crossness outfalls, and the recommendation was finally made that the present plant for chemical treatment be abandoned and that the London sewage be, first, settled to remove gross mineral matter; second, septicized for six hours; and, third, treated in single-contact beds of coke, 12 feet deep, at a rate of 5,200,000 gallons per acre, attained by four fillings per day (Clowes and Houston, 1904).

In 1894, as a result of the first Barking experiments, Dibdin installed seven experimental contact beds at Sutton, in Surrey. Here two important modifications of the contact system were introduced. In the first place, the sewage was subjected to successive treatments, first in coarse and then in fine-grained beds, now known as the "double contact" system. In the second place, after the process had worked well with chemical effluent, as it had done at London, the treatment of crude sewage was attempted. Beginning November, 1896, a double-contact system treating crude sewage was operated for the first time. The depth of the beds was 3 feet 6 inches, and the filling material burnt ballast, larger than three-eighths inch. Two fillings a day were made, giving a rate on each individual bed of 900,000 gallons per acre. The analytical results showed a reduction of oxygen consumed from 76 parts per million in the sewage to 26 parts in the effluent of the first bed and 10 parts in the effluent of the second bed (Dibdin, 1903).

The Physics and Chemistry of the Contact Bed. Dibdin's original idea was merely to accelerate the process of intermittent filtration so that small beds of coarse material might take the place of larger sand areas; and it was at first assumed that the fundamental processes involved were essentially the same.

Dunbar and Thumm (1902), in a beautiful series of experiments at Hamburg, have since shown that the reactions in the contact filter, as a result of the alternate aerobic and anaerobic conditions, follow a peculiar and characteristic course, and that while the

changes that go on in the empty period resemble those of the intermittent filter, they are complicated by a widely different set of reactions in the full period. Both physical and chemical changes play an important part. While the bed stands full, the solids in the sewage collect on the surface of the filling material and the soluble constituents are to a large extent adsorbed by the bacterial jelly with which the material is coated, or changed to insoluble colloids. The adsorption takes place in virtue of the general tendency exhibited by colloidal films to remove substances from contiguous solutions. Dibdin illustrates the removal of suspended matter by analogy with the adhesion of floating chips to larger bodies, and compares the adsorption of dissolved material to the removal of lead acetate by passage through a carbon filter. Dunbar offered striking evidence of the importance of this physical factor by a series of experiments in which he determined the minimum time necessary to produce purification. With a well-ripened filter he found that the oxygen-consumed value was reduced 83 per cent by five minutes' contact (Dunbar, 1908). This was confirmed by Frankland, who found that a value for oxygen consumed of 555 parts per million for raw sewage was reduced to 93 in five minutes. It was still 93 after thirty minutes and 49 after twelve hours.

The retention of the organic matter by a contact bed is purely physical; and some investigators (Bredtschneider, 1905) have even maintained that the whole action of the contact bed is a mechanical one. On this view the slow ripening of a filter, which clearly points to bacterial growth, would depend merely on the necessity for the formation of zoögleal films in which the adsorptive process could readily take place. It is quite clear, however, that the adsorbed material must afterward be chemically changed if any real purification of the sewage as a whole is to be effected. That this change is due to bacterial action is indicated by the fact that chloroform and mercuric chloride quickly put a stop to it (Dunbar, 1908). The chemical processes set up are much more complex than in the intermittent filter; for it has been shown by Dunbar and Thumm (1902), and Phelps and Farrell

(1905), that there are at least three distinct reactions involved, — nitrification during the empty period and hydrolytic splitting and denitrification during the full period.

In the nitrifying period the bacteria set in train processes essentially similar to those of the intermittent filter, which may be indicated by the following generalized formula:

Reaction 1. Nitrification:

$$(a)\ 2\,R \cdot N : R' + 3\,O_2 = R \cdot O \cdot R \cdot + 2\,R' : O + N_2O_3;$$
$$(b)\ N_2O_3 + O_2 = N_2O_5.$$

At the end of this period considerable quantities of nitrates are present in the filling material of the contact bed, and the amount of nitrates increases with the length of the period. When the bed is refilled the same action continues for a time. Soon, however, the supply of oxygen is consumed, active nitrification stops, and anaerobic putrefactions begin, causing hydrolytic splittings of the following type:

Reaction 2. Hydrolysis:

$$R \cdot N : R' + 2\,H_2O = R \cdot OH + HO \cdot R' \cdot NH_2.$$

At this stage the contact filter has the liquefying properties of the septic tank. There is a bacterial reduction of the nitrates to nitrites, and a formation of partly reduced nitrogenous bodies, primary amines, etc. This leads to a decomposition of the nitrites present and the liberation of gaseous nitrogen according to the following formula:

Reaction 3. Denitrification:

$$2\,HO \cdot R' \cdot NH_2 + N_2O_3 = 2\,R'(OH)_2 + 2\,N_2 + H_2O.$$

In intermittent filtration the nitrogenous constituents of sewage may be almost quantitatively converted into nitrites and nitrates. In the contact bed, however, it is obvious that the nitrates found in the final effluent give no true measure of the purification effected, since under ideal conditions the nitrates formed from half the nitrogen during the empty period, according to reaction 1, above, would be exactly used up in decomposing the other

half during the full period, according to Reaction 3. Dunbar and Thumm (1902) found as a matter of practical experience that the highest purification frequently accompanied the lowest nitrate content in the effluent. A part only of the nitrogen appears as nitrates, a part remains in a stable organic state and a considerable part is lost in gaseous form. Clark (1903) has pointed out that the loss of nitrogen amounts to from 38 to 50 per cent, and Phelps and Farrell (1905) found a loss of from 35 to 50 per cent.

From the chemical standpoint the decomposition of organic matter into free nitrogen, which takes place in this type of filter, is admirable in the completeness of the effect produced with a minimum consumption of oxygen. The alternation of two or more different processes in the same culture chamber is not, however, apt to lead to the best results. Constant conditions are generally essential to the most successful bacterial action; and in such a mechanism as a contact bed, which depends on a delicate adjustment between several groups of micro-organisms, there must often be serious impairments of efficiency due to an unbalancing of their proper equilibrium. It is therefore particularly unfortunate that practically nothing is known of the actual bacterial types concerned in the functioning of the contact bed.

Construction of Contact Beds. A contact bed is a water-tight basin, generally built of cement concrete. In England the attempt has been made to utilize simple excavations in clayey soil for this purpose; but leakage, settling and the working up of clay into the bed have generally introduced serious complications.

In making contact beds the ground is excavated to the depth of about four feet and the sides and bottom made water-tight — best by cement concrete six inches thick. The bottoms of the beds are channeled to receive drainpipes, or the channels themselves serve as drains, being covered with perforated slabs. The effluent from these drains passes into a main drain, which is so constructed that by the use of manholes and valves the effluent can be carried from a high-level to a low-level bed, or can be delivered directly into the effluent channel. The construction is such that the

effluent channels can be kept full of the purified sewage or completely emptied. The sewage carriers are so arranged that the sewage can be delivered upon any of the beds, either high or low level.

The size of a single bed should be so proportioned to the dry-weather flow of sewage that it will not contain, at most, more than two hours' flow, unless the sewage is applied from dosing tanks; for if it requires a long time to fill the beds the sewage that last enters is apt to be drawn from the bed before sufficient time has elapsed to produce the best results. For large plants, one-half acre has been fixed on in England as the maximum size.

The total volume of the beds to be provided for a single-contact system should be from one to one and one-half times the daily sewage flow, as the liquid capacity of a bed in operation will equal about one-third of its volume, and a bed can, as a rule, only receive two to three doses of sewage per day.

The usual depth allowed for contact beds is from three to four feet. Clowes and Houston (1904) reported from their London experiments that beds 3 feet, 5 feet and 13 feet in depth gave equally good effluents. Most other observers have found, however, that the better aeration obtained in shallow beds was distinctly beneficial. Studies at Exeter, in which samples were taken from taps placed at different depths in a contact filter, showed the best results at 3 feet below the surface, and at Manchester a 15-inch bed gave specially good results.

The underdraining of the contact bed must be carefully seen to, and the drains must be so laid and of such size that the bed can be completely emptied in one-half hour. To accomplish this, at some places, contact beds have been constructed with what are practically false bottoms made of tiles, separated from each other by a space of an eighth to a quarter of an inch. At other places the drains are laid in the cement concrete and covered with perforated tiles so as to prevent the falling of the filling material into the drain. Sometimes the drains are of pipe laid with open joints.

Almost any hard, non-friable material of the proper dimensions may be used for filling material in contact beds. Among the substances used in England are burnt clay, coal, coke, gravel, broken bricks, clinker, granite, sandstone, saggers (from pottery works) and furnace slag. Certain of the earlier experiments, like those of Clowes and Houston at Barking and those of the Massachusetts State Board of Health at Lawrence, indicated that

FIG. 75. Underdraining Contact Beds at Manchester.

coke was a particularly favorable material. Coal proved the best filling in other experiments. At Birmingham, for example, it was found that the purification, measured by reduction in oxygen consumed, was 64 per cent with broken stone, 71 per cent with slag and 93 per cent with coal (Bredtschneider and Thumm, 1904). Where coke was used with success, it was held to be in virtue of its rough and porous surface, while the advocates of coal supposed success was due to its smoothness. It seems improb-

able that the original surface structure of the fragments can make any particular difference when once they are covered by bacterial films and the suspended matter of the sewage. It is more likely that the differences observed were due to the chemical composition of the filling material. It has, for example, been clearly demonstrated that contact treatment is markedly promoted by the presence of iron in the filling material. This was made evident at Manchester (1901), and has been very fully worked out at Hamburg, as, for example, in the experiment tabulated below:

TABLE LVIII

EFFECT OF IRON-CONTAINING MATERIAL UPON CONTACT TREATMENT

Percentage Reduction in Oxygen Consumed.

(Dunbar, 1908.)

Months.	Gravel, 5–10 mm.	Gravel, 5–10 mm + 1% iron.
1	55·6	57·9
2	66.0	70.4
3	67·9	74·1
4	65·3	73·1

Iron-containing substances should thus be preferred if they happen to be available. Other materials may, on account of their chemical composition, prove specially undesirable. Raikes (1908) points out that slag containing lime or sulphur should be avoided, as liable to rapid disintegration. In general, however, the nature of the filling material is of slight importance and its selection will be largely controlled by local conditions of cost and convenience.

The size of the filling material used in a contact bed is, on the other hand, of prime importance. Both adsorptive efficiency and biochemical activity depend directly on the aggregate of the exposed surfaces of the fragments in the bed; and the smaller the fragments the greater will be the surface. Hering (1909) calculates that at Wilmersdorf a coke bed of 5-inch material has 25 square feet of bacterial surface to every cubic foot; at Birmingham, $1\frac{3}{4}$-inch slag and granite give 60 square feet of surface per

cubic foot of bed; and at Hanley, $\frac{3}{8}$-inch saggers give 135 square feet per cubic foot. Aside from this factor of bacterial surface, the bed of finer material exerts a considerably greater mechanical straining action and thus removes a larger proportion of suspended solids.

The following table from Dunbar gives a good idea of the relation between size of material and the purification effected:

TABLE LIX

EFFECT OF CONTACT TREATMENT WITH MATERIAL OF DIFFERENT SIZES

(Dunbar, 1908.)

Size of material.	Per cent purification, oxygen-consumed. Coke	Gravel.
2–3 mm.	70.2	61.8
3–5 mm.	69.0	61.8
5–7 mm.	64.6	57.0
7–10 mm.	62.5	56.6
10–20 mm.	51.0	46.5

In general it may be said that material of half-inch size or less will sometimes yield a stable effluent, while larger material will not. On the other hand, fine-grain beds clog very rapidly, and it is necessary to take out the material and wash it at more frequent intervals. The most economical balance between these two considerations has never been scientifically worked out, and different engineers have widely varying opinions. Clowes and Houston (1904), as a result of their London experiments, recommended the use of "walnut-size coke." In evidence before the Royal Sewage Commission, Fowler recommended one-eighth inch material, Cameron one-eighth to one-half inch, Frankland one-eighth to three-fourths inch, and Dibdin one-half to four-inch for first contact and one-sixteenth to three-eighths inch for second contact. Barwise (1904) suggests the use of coarser filling — three to five inch material for primary beds to treat septic effluent and one-half to one and one-half for secondary beds.

The amount of suspended matter in the liquid treated is of course one of the controlling features in deciding this point. The Royal Commission on Sewage Disposal (1908) summarizes its conclusions as follows:

"With a crude sewage containing 40 parts per 100,000 of suspended matter, the material will probably have to be from three inches upwards in diameter, and even then sludge will accumulate on the top.

"With a septic-tank liquor containing 8 to 10 parts per 100,000 of suspended matter, material of a diameter from three-eighths to five-eighths of an inch may probably be used effectively; while with a good precipitation liquor containing from 1 to 3 parts of suspended matter, the best results will probably be obtained from material as fine as one-fourth inch diameter.

"It is, however, impossible to make any but the most general statement as to the most suitable size of material for contact beds, as, in some cases, there may be special circumstances which affect the question, such as the character of the suspended matters or the smoothness of the filtering material. The sizes we have suggested are based on the evidence which we have received and also on our experience of contact beds at the places named in the following statement:" (see page 277).

In general the most successful arrangement for a contact plant will include a double-contact system with rather coarse material, over an inch in diameter, for the primary bed and with finer material for the secondary beds. The solid material of the sewage accumulates mainly in the first bed, the coarseness of which insures a reasonably long life, while the secondary fine bed provides for good purification.

In using the contact-bed system of treatment, the upper-level bed is filled as quickly as possible with sewage, usually in a half-hour; the sewage is, as a rule, allowed to remain two hours in the bed, and then run upon a lower-level bed, or, if sufficiently purified, directly into the effluent channel.

TABLE LX

DATA IN REGARD TO THE CONSTRUCTION OF ENGLISH CONTACT BEDS

(R. S. C., 1908.)

	Suspended matter in the liquor treated (approximate figures).	Nature of material.	Size of the material in the contact beds.
	Parts per Million		
Crude sewage			
Hampton..........	485	Clinker	Above 4″ diameter.
Leeds............	350	Clinker and coke	Above 3″ diameter.
Newton-le-Willows.	300	Clinker	Top 18″–½″ to ¼″ diameter. Body of filter—2″ to 1½″ diameter.
Withnell.........	200	Clinker	1″ to 1½″ diameter.
Maidstone........	140	Clinker	Above ¾″ diameter.
Settled sewage			
Oswestry.........	About 200	House coke	1½″ to ½″ diameter.
Halton...........	110	Clinker and pebbles	1″ to 2″ diameter.
Septic tank liquor			
Leeds............	180	Clinker	⅝″ to ⅜″ diameter.
Guildford........	160	Burnt ballast	½″ to 3″ diameter.
Hartley Wintney ..	150	Clinker	½″ to ⅛″ diameter.
Exeter (Main)....	About 140	Clinker	½″ to 1″ diameter.
Andover..........	120	Clinker	⅛″ to ½″ diameter.
Exeter (St. Leonards)..........	85	Clinker and coke	½″ diameter.
Slaithwaite.......	80	Clinker	Top foot — ⅜″ to ⅓″ diameter. Body of filter — ⅜″ to 1″ diameter.
Precipitation liquor			
Calverley.........	120 to 140	Clinker	¼″ to ½″ diameter.
Kingston (experimental beds) ...	20	Clinker and coke	Coke bed, ¼″ to 1″ diameter. Clinker bed, ¼″ to ⅜″ diameter.

Operation of Contact Beds. With regard to the operation of contact beds, the number of fillings is the first point to be considered. At Hamburg it was found that for single contact two fillings a day gave the best results, while for double contact six fillings of the primary beds and three fillings of the secondary beds were recommended (Dunbar and Thumm, 1902). In the Barking experiments it appeared that two fillings a day gave

better results than one; apparently a single filling does not maintain the bacteria at their maximum effectiveness. Birmingham experiments have indicated three fillings a day as effective, to be cut down to two if specially high purification is desired (Watson, 1903). At Crossness it was found that London sewage could be purified with as many as four fillings.

The distribution of fillings at regular intervals over the twenty-four hours does not appear to be a necessity. At Manchester contact beds were operated for two months with four six-hour cycles, and then for three months with four cycles in ten hours, followed by fourteen hours' rest. The results, as shown in the table below, were better by the second method.

TABLE LXI

RESULTS OF OPERATION OF CONTACT BEDS AT MANCHESTER, ENGLAND

(R. S. C., 1902.)

Mode of operation.	Analyses of effluent (parts per million).			
	Oxygen consumed in 4 hours at 80° F.	Nitrogen as —		
		Free ammonia.	Albuminoid ammonia.	Nitrates.
4 cycles in 24 hours................	29.0	16.8	1.5	2.6
4 cycles in 10 hours................	22.3	14.8	1.1	6.3

The duration of the full period may also vary. Dibdin adopted two hours, and this is perhaps the general English practice. In Germany, too, Schumburg and others advocate this period (Bruch, 1899). Harding at Leeds found that one hour gave inferior results, while four hours was no better than two (R. S. C., 1902). Roscoe and Cameron, on the other hand, advocate shortening the period to one hour (R. S. C., 1902). Perhaps the commonest cycle adopted is one which occupies eight hours, and allows one hour for filling, two hours for standing full, one hour for emptying and four hours for aeration.

Regularity in the operation of the contact bed is very essential

and this can often be most conveniently secured by the installation of automatic dosing devices. The filling and emptying of the beds, at definite intervals and in regular rotation, must be provided for in any device of this sort. The inflow to the beds is generally controlled by a series of float chambers, so connected by levers or air-lock siphons that when one bed is full the flow is diverted

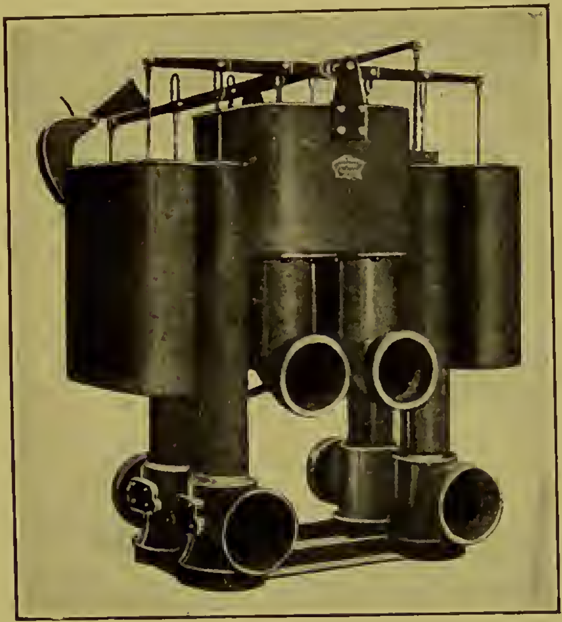


FIG. 76. Cameron's Alternating Gear for Dosing Contact Beds (courtesy of Cameron Septic Tank Company).

to the adjoining one. Sometimes the outflow is regulated by the same floats, so that when one bed is full and the one next in order begins to receive its dose the bed last treated begins at the same time to empty. One of the earliest of these devices designed by Cameron for use at Exeter is shown in Fig. 76. This gear controls four contact beds. The inlet and outlet valves of beds 1 and 2 are governed by a single lever, which is actuated by the floats in beds 3 and 4 and *vice versa*. Take the cycle at the time when bed 1 is full, 3 filling, 2 empty and 4 emptying. When bed 3 becomes full, the inlet of 2 and the outlet of 1 are opened.

The sinking of the float in bed 1 then closes the inlet of 3 and the outlet of 4 by the lever on the opposite side of the gear. Thus bed 3 stands full and bed 4 empty, 2 fills and 1 empties, during the next period. Proper adjustment of piping with a regular flow of sewage will give approximately a two-hour period for each process in the cycle.

This system has the defect that filling and emptying periods must be made as long as the periods of standing full and empty, which is undesirable. The whole cycle will vary in length with the rate of sewage flow. In many plants the arrangement is therefore modified, so that while the rotation of the beds is controlled by the inflow of sewage the effluent gates are governed by timed siphons designed to discharge after a definite period of contact. This is generally effected by filling the effluent siphon chamber from the bed by a small pipe which takes a definite time to deliver the contents of the chamber. Thus the filling and emptying of each bed may be made as rapid as desired and the standing full period may be fixed at two hours, any variations in the rate of flow being taken up by the more elastic and longer period of aeration.

In England the trend of opinion is generally adverse to automatic operation of sewage plants. The Royal Commission in its final report (1908) concludes: "Our own observations and the experience of others show that it is not possible to rely entirely on automatic apparatus for sewage works, although within certain limits it may be advantageously used with considerable saving of labor. In the case of large sewage disposal works, where men are always available, we consider that it would usually be inexpedient to provide an automatic plant. It is liable to get out of order, it does not adapt itself to variations in volume and strength of sewage, state of beds, etc., and generally it is preferable to work the beds by manual labor, where such labor is at hand." For small installations, on the other hand, some measure of automatic control is almost an essential; although the necessity for expert supervision should in no case be forgotten.

The distribution of sewage in small contact beds requires no

special provision. In large installations surface troughs are often provided to avoid lateral filling. At Manchester the sewage is discharged in radiating surface channels lined with fine screenings, which retain the bulk of the suspended matter and thus prolong the life of the beds themselves (Fig. 77).

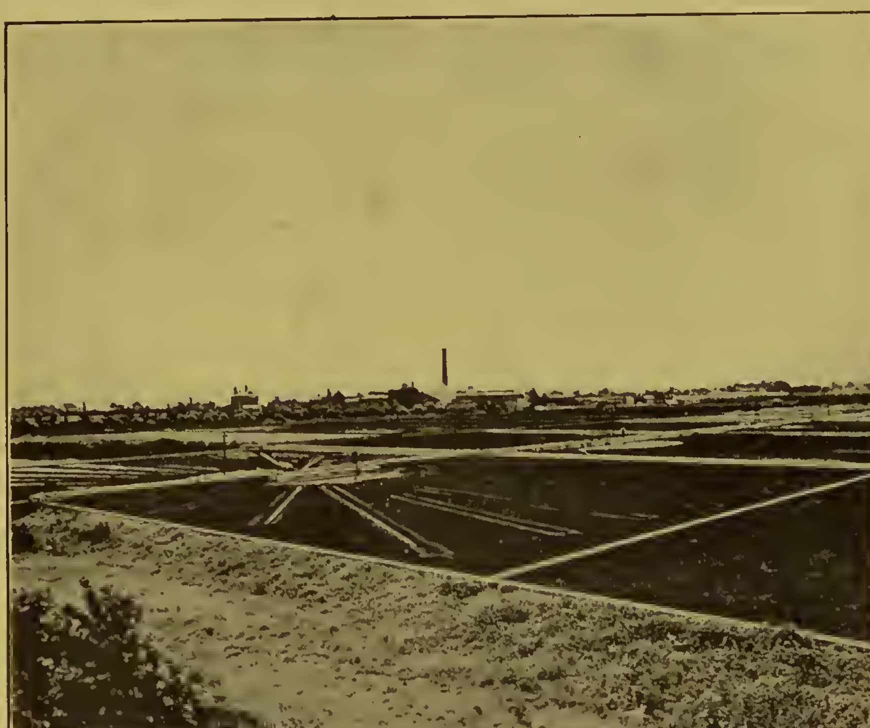


FIG. 77. Distribution of Sewage on Contact Beds at Manchester.

At Plainfield, N. J., and at Ballston Spa, N. Y., the sewage is delivered through one-half foot pipes laid about one foot below the surface (Fig. 78). Using this method the surface of the bed remains clean, and there is less danger of a nuisance being created from odors, or of pathogenic bacteria being carried away from the bed by winged insects.

In appearance the contact effluent is fairly clear, but usually rather dark in color. In regard to the chemical composition, the purification effected by single and double contact treatment is

TABLE LXII

RESULTS OF SINGLE AND DOUBLE CONTACT TREATMENT

Parts per million. (Winslow and Phelps, 1906.)

	Solids.						Nitrogen as —												Oxygen consumed in 4 hours at 80° F.*		
	Total.			Suspended.			Free ammonia.			Albuminoid ammonia.			Nitrites.			Nitrates.					
	Sewage.	First contact.	Second contact.	Sewage.	First contact.	Second contact.	Sewage.	First contact	Second contact.	Sewage.	First contact.	Second contact.	Sewage.	First contact.	Second contact.	Sewage.	First contact.	Second contact.	Sewage.	First contact.	Second contact.
Lawrence: **																					
Nos. 137–163	....	....	....	....	....	....	36.3	24.3	12.0	5.9	2.4	1.4	....	....	....	....	8.3	14.6	42.6	21.2	10.3
Nos. 137–164	....	....	....	....	....	....	36.3	24.2	8.6	5.9	2.4	1.2	....	....	....	....	8.3	21.7	42.6	20.4	9.4
Aylesbury †	1,190	1,020	920	367	112	0	56.4	42.7	23.6	6.5	4.0	1.9	....	.3	.9	....	.6	13.3	57.5	35.1	18.4
Blackburn ‡	1,080	602	596	474	60	3	23.3	17.3	11.4	2.8	1.7	1.0	....	.1	.2	....	.4	1.1	35.8	23.0	12.6
Leeds: §																					
Nos. 1 and 2	1,670	1,080	980	584	165	27	20.2	12.9	3.7	9.5	3.7	.9	....	....	....	....	....	5.6	115.0	36.0	7.0
Nos. 3 and 4	1,690	1,180	1,050	614	180	47	23.8	10.8	6.8	11.3	3.4	1.3	....	....	....	....	....	3.7	126.0	30.0	9.0
Nos. 5 and 6	1,780	1,170	1,020	661	196	47	24.1	14.5	7.8	11.8	4.7	1.7	....	....	....	....	....	1.8	132.0	42.0	11.0
Leicester §§	1,320	1,070	1,020	296	73	23	....	....	....	10.6	4.2	2.5	....	....	....	.6	.9	4.6	83.9	33.4	10.7
Manchester: ‖																					
Beds A and D	....	....	....	....	....	....	24.0	9.1	3.7	3.7	1.4	.6	....	.4	.2	....	5.6	10.6	83.9	23.1	7.5
Beds C and D	....	....	....	....	....	....	24.0	8.5	3.6	3.7	1.3	.6	....	.3	.2	....	6.3	10.8	83.0	19.6	7.5
Sutton ‖‖	1,630	895	1,000	824	45	6	94.2	32.6	8.3	7.3	2.9	1.4	....	1.5	.7	....	4.1	19.7	56.9	21.9	9.2

See Notes on page 283.

fairly represented by the data collected from various sources and tabulated and plotted in diagrammatic form by Winslow and Phelps (1906) (Fig. 79). The first contact removes somewhat more than half of the organic constituents of the sewage, as measured by oxygen consumed and albuminoid ammonia, and two-thirds or more of the suspended solids, while the second contact effects almost as great a purification on the first-contact effluent. Aylesbury and Blackburn show the worst results among the English plants as far as ratio of purification is concerned. It will be noticed that these are the weakest sewages, and in all sewage treatment the last fractions of organic matter are the most difficult to remove. Except at Lawrence the nitrate content of the effluent is rather low, notably at Leeds and Leicester.

NOTES TO TABLE LXII

* At Lawrence: 2 minutes' boiling.

** Lawrence: No. 137–163, run for 18 months during 1901–1902; area of beds, $\frac{1}{20000}$ acre each; first contact, broken stone $\frac{1}{2}$ to 1 inch, taking raw sewage, rate 0.9; secondary bed, fine coke, rate 0.7 (Fuller, 1905). No. 137–164, run for 6 months during 1901, like No. 137–163, except that coke was considerably " finer " in secondary bed (Fuller, 1905).

† Aylesbury: Experiments during first 6 months of 1898, weekly analyses; no further data given (R. S. C., 1902).

‡ Blackburn: No further data (R. S. C., 1902).

§ Leeds: Nos. 1 and 2, experiments during October, 1898–October, 1899; beds about $\frac{1}{8}$ acre in area; primary contains 5 feet of coke over 3 inches in diameter, receives raw or settled sewage at rate of 0.8; secondary, 6 feet coke, $\frac{3}{16}$ to $1\frac{1}{2}$ inches, rate like primary. Nos. 3 and 4, experiments during November, 1898, to June, 1900; beds each about $\frac{1}{8}$ acre in area and 3 feet deep; primary bed filled with clinker $\frac{1}{2}$ to 1 inch, and secondary $\frac{1}{16}$ to $\frac{1}{2}$ inch; rate during first 6 months, 1.1 (settled sewage), and during remainder of period 0.3 (raw sewage). Nos. 5 and 6, experiments during March to November, 1899; beds each $\frac{1}{8}$ acre in area; primary bed, 3 feet of clinker, 1 to 2 inches; secondary bed, 3 feet of clinker, $\frac{1}{16}$ to $\frac{1}{2}$ inch; rate, 0.8, raw sewage on each bed; 4 hours' contact during a portion of the time (Leeds, 1900).

§§ Leicester: Process No. 14; experiments during November, 1898, to July, 1899; primary beds, $\frac{1}{40}$ acre in area, filled with clinker $1\frac{1}{4}$ to $2\frac{1}{4}$ inches, $4\frac{1}{2}$ feet deep; secondary bed, $\frac{1}{65}$ acre in area, filled with 3 feet of clinker, $\frac{1}{8}$ to $\frac{1}{2}$ inch; rate, 1.6 on each bed, septic sewage (Leicester, 1900).

‖ Manchester: Experiments during January 4, 1900, to March 27, 1901; A and C were primary beds, the combined effluents of which were run on to D, area of each bed, $\frac{1}{77}$ acre on the surface; slope inward, 2 : 1. A contains 3 feet of clinker $\frac{1}{8}$ to $\frac{3}{4}$ inch; septic sewage at rate of 0.3. C contains 3 feet of clinker $\frac{1}{4}$ to $\frac{3}{4}$ inch; septic sewage at rate of 0.4. D contains 3 feet of clinker $\frac{1}{8}$ to $\frac{1}{2}$ inch, rate, 0.7 (Manchester, 1901).

‖‖ Sutton: Experiments from October, 1897, to August, 1898, coarse bed 3 feet deep, of burnt ballast; 5 beds of burnt clay; rate in each bed, 1 (R. S. C., 1902).

Rates expressed in million gallons per acre per day.

The effluent of the first-contact process, as is obvious from the analyses in the table above, generally contains too much organic matter to be considered satisfactorily purified, while two successive treatments produce, as a rule, an effluent which is non-putrescible. Double contact is also more effective than a single treatment at half the rate, as indicated by the Manchester results

FIG. 78. Distribution of Sewage on Contact Beds at Plainfield, N. J. (courtesy of F. E. Daniels).

tabulated below; and a double-contact system, with coarse primary and fine secondary beds, is probably the form best adapted for applying this general method of sewage treatment.

TABLE LXIII

RESULTS OF DOUBLE AND SINGLE CONTACT TREATMENT AT MANCHESTER, ENGLAND

Parts per million. (Manchester, 1904 a.)

	Nitrogen as —			Oxygen consumed in 4 hours at 80° F.
	Free ammonia.	Albuminoid ammonia.	Nitrates and nitrites.	
Septic effluent	25.8	2.5	....	70
First contact	14.5	1.2	0.5	22
Second contact	4.1	.5	8.5	7
Septic effluent	31.0	3.5	....	80
Single contact (one-half rate)	13.3	1.3	4.3	16

In some cases an attempt has been made to improve the effluent still further by adding a third contact treatment, as, for instance at Hampton Court. What may be expected from this

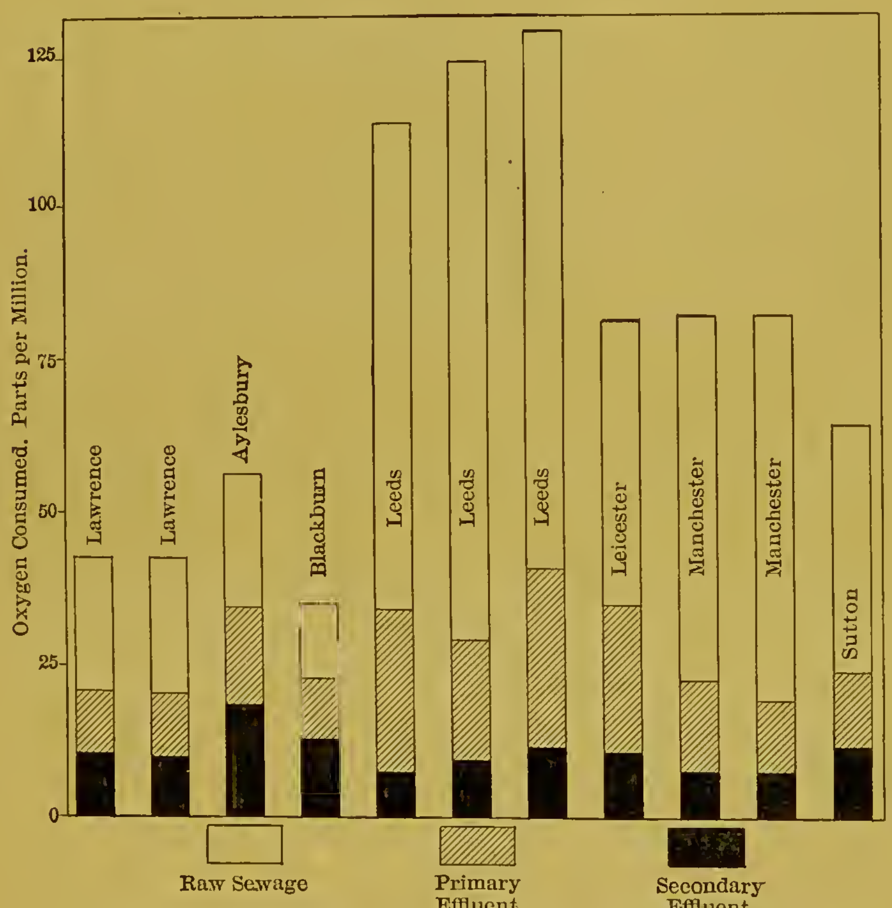


FIG. 79. Purification Effected by Single and Double Contact Beds.

plan is indicated in the table on page 286. In general, the improvement in successive treatments progressively lessens, so that the results obtained are scarcely commensurate with the cost. The head required for successive contacts also introduces a serious factor.

TABLE LXIV

RESULTS OF TRIPLE-CONTACT TREATMENT

(Parts per million)

	Solids.		Nitrogen as —			Oxygen consumed in 4 hours at 80° F.
	Total.	Sus-pended.	Free ammo-nia.	Albumi-noid am-monia.	Ni-trates.	
Eastry (R. S. C., 1902).						
Sewage	1,550	1,070	25.5	12.8	4.6	123.0
Bed 1	1,460	107	22.0	3.0	1.9	50.5
Bed 2	1,340	85	12.4	2.4	2.1	25.4
Bed 3	1,360	21	4.8	1.2	7.4	17.2
Leeds (Leeds, 1900).						
Sewage	1,760	632	27.6	12.4	......	127.0
Bed 1	1,250	274	18.6	7.1	......	62.4
Bed 2	1,060	113	13.5	5.1	......	39.6
Bed 3	1,030	110	9.7	3.5	2.0	27.5

The rate of filtration on contact beds, which is usually expressed in relation to the superficial area, is of course a function of the depth and the number of fillings. It would be more reasonable to measure contact rates in such units as acre-yards, which take account of depth. For uniformity with sand and trickling filters, however, the unit of superficial area is convenient. With a bed 3 feet deep and an open space of 33 per cent, which is a liberal estimate for a matured filter, two fillings a day would equal a rate of 650,000 gallons per acre per day and three fillings a rate of about 1,000,000 gallons. In practice, necessary rests and loss of capacity being taken into account, three fillings of a 3-foot bed will not amount to a rate of more than 800,000. At Barking in 1898, Clowes and Houston (1904) obtained with one filling rates of 600,000 for coke and 500,000 for ragstone, and in 1899 with two fillings the rates were increased only to 700,000. Watson (1903) considers 400,000 to 600,000 the best rate attainable, even when the sewage has been previously subjected to septic treatment. The table on page 287, compiled from Watson's Birmingham lecture and from the testimony before the Royal Sewage Commission, indicates the rates which have been obtained in actual operation or in experiments on a practical scale.

TABLE LXV

CONTACT-FILTER RATES

(Watson, 1903; Martin, 1905.)

Single contact.			Double contact.		
Place.	Depth (feet).	Rate (million gallons per acre per day).	Place.	Depth (feet).	Rate (million gallons per acre per day).
Manchester	3.3	.6	Burnley	3.0	.3
Birmingham	4.5	.6	Leeds	5.5	.6
Croydon	3.7	.8	Blackburn	5.5	.8
Exeter	5.0	1.0	Sheffield	3.3	.8
Sutton	3.5	1.0	Carlisle	4.0	1.1
London	3.0	1.2	Sheffield	3.3	1.2
Leeds	4.5	1.4			

When a double-contact system is used, the area must naturally be increased, so that the net rate on the total area for a system which will yield a stable effluent will not be over 500,000 gallons per acre per day.

In general it may be concluded that contact beds will purify sewage at about eight times the rate ordinarily attained by the use of intermittent filters. The effluent will not be so highly purified as in the latter case, but it will be sufficiently stable for all practical purposes. It must be remembered, however, in making such comparisons, that the contact bed has much less flexibility than a sand filter in dealing with exceptionally high flows of sewage. Where large amounts of storm water must be dealt with, special provision must be made in the case of the artificial process.

The Loss of Capacity in Contact Beds. One of the most serious problems in regard to the operation of contact beds is the progressive loss of capacity which appears to be an inevitable feature of the process. If a bed were filled with perfect spheres of uniform size, its open space or water capacity would be 26 per cent of its entire cubic contents. In beds built of ordinary irregular materials this original value varies from 30 to 50 per cent. In

the course of operation, however, it falls to 20 or even to 10 per cent, and thus reduces the rate of sewage treatment to a half or a quarter of its original value. The rate of capacity loss varies, of course, with the amount of suspended matter in the treated liquor and varies inversely with the size of the filling material. The table below, from the Hamburg experiments, brings out both these points, the fine slag in primary beds showing a greater loss than coarse primary beds or secondary beds of fine material.

TABLE LXVI

REDUCTION IN CAPACITY OF HAMBURG FILTERS

(Dunbar and Thumm, 1902.)

Material.	Loss in capacity (gallons per million gallons filtered).	Material.	Loss in capacity (gallons per million gallons filtered).
Single contact.		*Second contact.*	
Slag, $\frac{1}{8}$- to $\frac{5}{16}$-inch	1,330 1,680	Slag, $\frac{1}{8}$- to $\frac{5}{16}$-inch	420
		Gravel, $\frac{1}{8}$- to $\frac{5}{16}$-inch	700
Coke, $\frac{3}{8}$- to $1\frac{1}{8}$-inch	340	Coke, $\frac{1}{8}$- to $\frac{5}{16}$-inch	630
Gravel, $\frac{3}{8}$- to $1\frac{1}{8}$-inch	280	Slag, $\frac{3}{8}$- to $1\frac{1}{8}$-inch	340
Slag, $\frac{3}{8}$- to $1\frac{1}{8}$-inch	170	Coke, $\frac{3}{8}$- to $1\frac{1}{8}$-inch	360
Brick, $\frac{3}{8}$- to $1\frac{1}{8}$-inch	440	Gravel, $\frac{3}{8}$- to $1\frac{1}{8}$-inch	460
		Gravel and iron, $\frac{3}{8}$- to $1\frac{1}{8}$-inch	650

The course of this progressive loss of capacity is significant. In the first place, there is an apparent loss, due to the saturation of the surface and pores of the material, which should not properly be included here at all, but which may prove misleading if the original loss of capacity is determined by measuring the fluid required to fill the bed, instead of first filling it and then measuring the effluent. Aside from this false loss of capacity a true loss begins almost as soon as a bed is put in service, and goes on with considerable rapidity, at a rate of several hundred gallons' loss of capacity for every million gallons of sewage treated. After some six months of use the capacity of the beds decreases much more slowly and may remain fairly constant at perhaps one-half its original value.

The causes of this capacity loss may be grouped conveniently under three main heads: *a*, breaking down and settling together of material with the consequent impairment of drainage; *b*, growth of organic films; *c*, deposition of the insoluble constituents of the sewage.

The decrease in capacity is first, then, due to the breaking down of uniform materials into pieces of more varied size, which become more closely packed together. The amount of this loss may be measured by the space left by the settling over the top of the material. At Pawtucket, R. I., it was estimated that about one-third of the total capacity loss in eighteen months was due to this factor. Such loss may be avoided, to a great extent, by the use of compact and permanent filling. This was shown very clearly in the Hamburg experiments, in which it appeared that slag, while giving as good analytical results as coke and gravel, showed appreciably less loss of capacity. In England a great deal of experience has been accumulated in regard to the settling together and breaking down of friable material. At Leeds Colonel Harding and Mr. Harrison examined the material in a primary bed of coke after one year's use and found that 45 per cent of it passed a $1\frac{1}{2}$-inch screen, although it had originally been over three inches in diameter. At Manchester the rescreening of clinker filling showed that 8–10 per cent had become too fine for use; and at Newton-le-Willows and Andover, after 4–6 years' use, about 25 per cent of the furnace clinker used had been reduced to fine fragments (R. S. C., 1908). Many English beds are, however, still built of friable stuff.

The second loss, due to the growth of the organic films within the bed, is a loss that cannot well be prevented, for it is intimately correlated with the purifying process. As Dr. Fowler has stated: "This (growth of organisms) is at once the cause of increased efficiency in the bed, and of loss of capacity. On examining the material of a contact bed in active condition, every piece is seen to be coated over with a slimy growth. If this is removed, it soon dries to a stiff jelly, which can be cut with a knife. Under the microscope masses of bacteria and zoöglea will

be found to be present. If placed in a tube containing air, and connected with a manometer, the jelly will rapidly absorb all the oxygen and produce carbon dioxide" (R. S. C., 1908). Dunbar gives the figures quoted below to illustrate the progressive improvement in the efficiency of contact beds as they grow older and as the surface films increase:

TABLE LXVII

IMPROVEMENT IN AMMONIA ABSORPTION IN A CONTACT FILTER

(Dunbar and Thumm, 1902.)

Months at work.	Reduction in ammonia (per cent).	
	Single filling.	Double filling.
1	9.1	14.6
2	34.6	30.9
5	35.2	23.0
8	47.4	41.3
10	43.0	41.2
14	40.5	41.6

The losses due to growth and breaking down of material are almost independent of the character of the liquid filtered. Dunbar treated coke contact filters with various substances (134 fillings in four months) and found with tap water a reduction in capacity from 48 to 40 per cent; with tap water plus 1 per cent urine, from 47 to 37 per cent; with unfiltered sewage, from 48 to 37 per cent; with filtered sewage, from 48 to 40 per cent; and with sewage precipitated with lime, from 44 to 36 per cent (Dunbar and Thumm, 1902).

The loss of capacity due to organic growths can largely be made good by allowing the beds to stand empty for two weeks or more. Under these conditions the organic films dry up, shrivel and disintegrate, and the original capacity of the bed is restored, to a considerable extent, but never wholly. The efficiency of this process is well shown by the results tabulated on page 291, obtained at York and Leeds:

TABLE LXVIII

LOSS IN CAPACITY OF CONTACT BEDS AND RECOVERY BY RESTING

NABURN DISPOSAL WORKS, YORK (YORK, 1901).

	U. S. gallons.	Per cent open space.
Bed No. 1.		
Cubic capacity	55,200	100
Initial liquid capacity	22,300	40
After 90 days' work	11,200	20
After 14 days' rest	16,400	30
After 42 days' work	11,500	21
KNOSTROP SEWAGE WORKS, LEEDS (LEEDS, 1900).		
Bed No. 7, single contact.		
Cubic capacity	222,000	100
Initial liquid capacity	66,800	31
After 226 days' work	25,900	12
After 74 days' rest	64,200	30
After 184 days' work	30,700	14
Bed No. 8, single contact.		
Cubic capacity	113,000	100
Initial liquid capacity	35,400	31
After 185 days' work	12,800	11
After 50 days' rest	32,300	28
After 203 days' work	11,800	10

There remains, of course, the loss of capacity due to the deposition of suspended solids from the sewage and the change of soluble into insoluble colloids; and this is in some respects the most serious factor of all. It cannot be entirely avoided; and it can only be minified by removing as much as possible of the suspended matter by some preliminary treatment. The relation between capacity loss and suspended solids is well shown in the table below from results tabulated by the English Royal Commission:

TABLE LXIX

LOSS OF CAPACITY IN CONTACT BEDS

(R. S. C., 1908.)

Place.	Suspended matter in applied liquor. Parts per million.	Age of of bed. Months.	Average daily fillings.	Total million gallons treated.	Capacity loss Gallons.	Capacity loss. Gallons per million gallon.
Guildford	159	46	2.0	224	108,794	486
Exeter (main works)	125	37	1.6	2056	750,750	365
Andover	111	38	1.2	96	79,600	828
Exeter (old works)	82	109	1 6	196	39,670	202
Slaithwaite	71	84	2.0	226	33,755	149

With the exception of Andover, the loss of capacity per million gallons was approximately proportioned to the amount of suspended matter put upon the beds. The high rate of loss at Andover was accounted for by the fact that the material in the beds disintegrated so badly as to cause a considerable sinking of the surface level.

These considerations have led English engineers to make unusual efforts to remove as much suspended matter as possible, before contact treatment, by sedimentation or chemical treatment. The purification of crude sewage is rarely attempted; but septic or chemical effluent may be handled with reasonably good results. In the Barking experiments (Clowes and Houston, 1904) the capacity of two primary coke beds fell in ten months with crude sewage from 69–70 per cent to 18–20 per cent. Secondary beds showed only a reduction from 62 to 55 per cent (coarse) and from 53 to 44 per cent (fine). The stone beds lost about 1 per cent of their original liquid capacity per week. A series of experiments with septic effluent followed, in which after the first loss a capacity of about 30 per cent was constantly maintained. At Leeds it was found that beds taking septic effluent showed much higher capacities than those which received crude sewage. Similar conclusions were drawn by the experts at Manchester, although the experiments made with crude sewage were not exhaustive. The capacity of beds treating septic effluent decreased during the first three months and then remained fairly constant at about 33 per cent. At Burnley, with septic effluent, the capacity of contact beds fell from 44 to 19 per cent; at Exeter it fell from 39 to 28 per cent; and at Leicester from 49 to 29 per cent. At Sutton a minimum of 21 per cent was reached (R. S. C., 1902).

It was hoped that by careful preliminary treatment for the removal of suspended solids, and by resting to disintegrate organic growth, contact beds might be used more or less indefinitely, with a capacity of 20–30 per cent of their original cubic contents. Dunbar and the other German investigators from the first took the view that renewal was inevitable, and that it might

be economical not to attempt to keep clogging down to a minimum, but to treat crude or roughly clarified sewage on material as fine as seemed desirable, removing and washing the contents of the bed whenever the capacity fell to 20–25 per cent. This might be required two or three times a year (Dunbar and Thumm, 1902). The Prussian commission at Berlin came to a similar conclusion (Bruch, 1899).

Fig. 80. Washing Contact Material by Hand at Hamburg (copied by permission from Dunbar, 1908).

Experience in the main has justified this position, and has shown that with all care in construction and operation, even with septic tank effluents contact beds require renewal after a comparatively short period of time. The application of chemically treated sewage seems somewhat more promising according to the experience obtained at Leeds. At Manchester the life of the beds has proved to be about five years. The simple plant used for washing the material at Hamburg is shown in Fig. 80. Similar arrangements for hand washing are in use in many of the smaller English towns. At Manchester

the process is more elaborate, being conducted on a large scale. "The method employed is to pass the material from a sump to a jigging-screen of $\frac{1}{4}$-inch mesh, over which are fixed a series of horizontal water sprayers. The material rejected by this screen is further graded by passing over a fixed 2-inch mesh screen. The material which passes through the $\frac{1}{4}$-inch screen falls onto an inclined fixed screen of $\frac{1}{8}$-inch mesh. All material above $\frac{1}{4}$-inch is replaced in the primary filters; that between $\frac{1}{4}$-inch and $\frac{1}{8}$-inch will be used for the surface of the secondary contact beds, when these are constructed. Settled sewage is the liquid used in the washing. The total cost of removing, washing, screening and replacing in beds and making up to original level with new material at Manchester is 1*s.* 6*d.* per cubic yard" (R. S. C., 1908).

In general the cost of washing and renewal at the English contact plants has ranged between 25 and 50 cents per cubic yard. The material lost amounts to 20–25 per cent of the original filling; but the washed material is better than it was at first, as the softer and more friable portions have been eliminated.

Experience with Contact Beds at Manchester and Other English Cities. The history of sewage treatment at Manchester is typical of most of the manufacturing cities of England. Until 1889 all the sewage of the city was discharged untreated into the four streams and rivers that flow through Manchester, finding its way ultimately into the Irwell. In that year Parliamentary power was obtained to divert and treat the sewage, and $95\frac{1}{2}$ acres of land, subsequently increased to $165\frac{1}{2}$ acres, were acquired at Davyhulme, about five miles from Manchester, for the erection of works. The scheme involved $36\frac{1}{2}$ miles of intercepting and other sewers, a system of eleven tanks with a total capacity of 12,000,000 gallons for chemical treatment, a plan for dealing with the resulting sludge by eight filter presses, pumping engines, sludge wells, etc., and 36 acres of land for further treatment of the effluent from the precipitation tanks.

The works were completed in 1893. The sewage as it came to the plant was screened, lime and sulphate of iron was added, and the liquid was then passed through the precipitation tanks.

It was the original intention to discharge this effluent on the 36 acres of underdrained land, but this was never actually done, and the effluent, or at least the greater part, was discharged directly into the ship canal. The result was unsatisfactory from the very first; an effluent could not be obtained from the chemical treatment which would answer the requirements of the English provisional standard, or which would not undergo secondary decomposition.

Chemical precipitation for the treatment of Manchester sewage having thus been shown to be unsatisfactory, it was suggested that the effluent from the precipitation plant be purified by irrigation; but as this would have required 1300 acres, exclusive of the roads, carriers and the usual adjuncts of a sewage farm, it never received very favorable consideration.

In 1895 Sir Henry Roscoe suggested that the effluent from the precipitation tanks should be purified by treatment on bacterial contact beds, similar to the method tried experimentally by Dibdin at Barking; and by his advice two small bacteria beds, afterwards increased to four, were built, and an elaborate series of experiments under the immediate direction of Mr. Frank Scudder were undertaken. The beds were 18 feet long and 12 feet 6 inches wide and 4 feet deep, and were filled to the depth of 3 feet with the filling material, — one with coke, one with cinders, one with burnt clay and one with coal. The result of these experiments was, briefly, that cinders, on the whole, were found to be the best material, and that an effluent answering to the English standard and one that would not undergo subsequent putrefaction could be obtained by treating the effluent from the precipitation process at the rate of about 700,000 gallons per acre per day on prepared bacterial single-contact beds.

While the experiments under Mr. Scudder were still being carried on, Mr. Meade, the city surveyor, though expressing his confidence in the results that had been obtained, presented to the Committee on Drainage, March 1, 1896, a plan for carrying the effluent from the precipitation tanks $15\frac{1}{2}$ miles in a covered conduit to the tidewaters of the Mersey at a cost of about $1,300,000

and recommended the plan for adoption as solving for all time the sewage problem. This plan, however, was not accepted by the city of Manchester, and in May, 1898, a commission was appointed, consisting of Baldwin Latham, Esq., engineer; Dr. Percy L. Frankland, biologist, and Dr. W. H. Perkin, Jr., chemist, to advise regarding the three plans above mentioned, or on any "alternative scheme which may be put before them or which they may themselves recommend."

The investigations which were undertaken by the commission were on a sufficiently large scale to be classed, not as laboratory but as field experiments; and it is on this account as well as on account of the thoroughness and ability with which the work was carried on under the direct charge of Gilbert J. Fowler, that they were of the greatest practical importance.

The results obtained convinced the commission that both the dry and storm weather flow could be successfully treated by a combination of the septic tank treatment with contact beds, and in October, 1899, they recommended the double-contact method for the treatment of Manchester sewage. Work was begun almost immediately and the first-contact beds were put into service in 1901.

In 1909 the general plan of the sewage plant was as follows: Four settling tanks, capacity of each 1,630,000 gallons; 12 septic tanks, total capacity of 19,500,000 gallons; 92 first-contact beds, each 0.5 acre superficial area, constructed of cement concrete and filled with furnace clinker (rejected by a 1.6 inch mesh passed by a 0.25 inch mesh) to a depth of 40 inches. Besides these 92 first-contact beds there is an area of 27 acres divided into 29 storm-water contact beds. In these beds the filling material, 2.5 feet of unscreened furnace clinker, rests generally upon a heavy clay marl, though where needed a layer of cement concrete has been laid down. They are designed to operate at a rate of 3,000,000 gallons per acre per 24 hours. The original plan also included 52 half-acre second-contact beds similar in construction to the first-contact beds, except that the filling material was to be somewhat finer.

At the present time the whole of the sewage is treated in septic tanks, and in dry weather is carried to the contact beds; in wet weather the excess is treated on the storm beds.

The amount treated on the contact beds averages 600,000 gallons per acre per day. The effluent is, however, not perfectly satisfactory, often undergoing secondary putrefaction. At the present time 600,000 gallons of the effluent of the first-contact beds is passed on to the one second-contact bed (0.5 acre area) that has been built. The effluent from this second-contact bed has so far been nonputrescible, and it is planned in the near future to treat all of the effluent from the first-contact beds on secondary beds. Allowing that a suitable effluent can be thus obtained, it requires for the satisfactory treatment by double contact of 600,000 gallons per day 1.5 acres, or one acre for 400,000 gallons, and this is the figure that is now given by Mr. Fowler.

The general result at Manchester has justified the original design of the plant, and for five years the works have treated by contact filters 90 per cent of the total flow, with an average purification, as shown below, of over 70 per cent, and have yielded an effluent often less putrefactive than the water of the ship canal.

TABLE LXX

EFFECT OF TREATMENT OF MANCHESTER SEWAGE ON CONTACT BEDS

Results for the Year ending March 30, 1904

(Parts per million.)

Source of sample.	Free ammonia.	Albuminoid ammonia.	Nitrogen as nitrites and nitrates.	Oxygen consumed.
Crude sewage	22.7	5.2	....	105.4
Septic tank effluent	26.3	3.7	....	93.5
Primary contact-bed effluent	16.2	1.5	2.7	27.2

Average purification: On albuminoid ammonia basis, 71 per cent.
On oxygen-consumed basis 73 per cent.

During the first years of the working of the plant, owing largely to the amount of suspended matter in the effluent from the septic

tank, Mr. Fowler anticipated that it would be necessary to wash and replace the filling material, and in 1907 the beds were clogged to such a degree that it was considered desirable to remove the filling material, wash it, and, after replacing the loss of material, to refill the beds. This was done at an average expense of about 31 cents per cubic yard. If in the original design chemical treatment had not been replaced by septic tank treatment, the washing of the filling material after as short a period as five years would probably not have been necessary.

Cost data of great value have been accumulated at Manchester. The amount of money spent, and to be spent in the immediate future, for the construction, including the original works for precipitation, will aggregate about three million dollars. The first cost of building the contact beds, including excavation, underdraining, laying concrete and putting filling material in place, was $13,000 per acre. The filling material in place cost 87 cents per cubic yard. The cost of removing filling material, washing and replacing it, and making good the fine material lost in washing, brings the total cost of renewal up to about 50 per cent of the first cost of filling. The general operating costs of the plant are tabulated below from the figures for 1906–1907:

TABLE LXXI

WORKING COSTS OF TWELVE PRIMARY CONTACT BEDS AT MANCHESTER

(Eng. News, 1908)

Average number of fillings per bed	2,690
U. S. gallons septic effluent treated on 12 beds (6 acres)	4,613,624,000
Total maintenance cost	$4,087.26
Total renewal cost (40½ cents per cubic yard)	13,705.20
Maintenance cost per million gallons filtered	.89
Renewal cost per million gallons filtered	2.97
Actuating valves per million gallons filtered	.25
Total working cost per million gallons filtered	4.11

The average per capita cost of sewage treatment for the five years 1903–1907, including sludge disposal, filtration, all maintenance and renewals and all general expenses, but with certain receipts deducted, has varied from 12 to 18 cents.

About 1900 a number of other contact systems were installed or recommended in English cities. Huddersfield had a serious

problem in the presence of large amounts of industrial waste from the scouring and dyeing of wool; but it was shown in a series of experiments carried out between 1898 and 1900 by J. L. Campbell that chemical treatment, sedimentation and contact treatment would solve the difficulty satisfactorily (R. S. C., 1902). At Oldham studies carried out by J. B. Wilkinson from 1898 to

FIG. 81. Filling Contact Beds at Manchester.

1900 led to the adoption of sedimentation and single-contact beds (R. S. C., 1902).

A valuable review of existing conditions in 1904 by M. N. Baker (1904) described septic tank and contact filter plants in operation at Exeter, Yeovil, Barrhead and Burnley, as well as at Sutton, Manchester and Oldham. In general, the tendency has been of late away from the construction of contact beds on account of

their short life, and in view of the higher rates attained with trickling filters.

At Sheffield, however, a city of nearly half a million population, with a sewage flow of fifteen million gallons, a system of single-contact treatment is now under construction. Since 1886 chemical treatment in large settling tanks has been the chief method of purification. The new plant is thus described by Mr. H. W. Clark (1908):

" Sixteen settling tanks are being constructed, each with a capacity of 1,000,000 gallons, and chemical treatment is to be omitted. Thirty acres of contact beds in half-acre sections are being constructed. All these contact beds are most solidly built with brick wall, concrete bottoms 6 inches thick and brick and concrete channels. The beds are to contain 4 feet in depth of clinker over the underdrains, and the main underdrains are being built of concrete below the floor of the filter with tile coverings, and side drains 10 feet apart entering these are laid on the concrete flooring. The material of the bed is to be graded clinker 3 to 6 inches in diameter and becoming finer towards the top, the upper six inches to be constructed of clinker not more than $\frac{1}{4}$ or $\frac{3}{8}$ inches in diameter. The sewage is to pass to these contact beds through a channel built between each set and will enter the bed through a 2-foot pipe to a chamber in the center of the bed, where it will rise and overflow to a second circular chamber 15 feet in diameter. From this it will pass over the surface of the beds in channels formed of the fine surface coke. The building of contact filters at Sheffield is a result of the operation, for ten years, of large experimental contact filters treating 1,000,000 gallons of sewage daily. These filters produced an effluent equal to the requirements of the Local Government Board, and it is stated that the filtering material was never cleaned or renewed during their period of operation."

Contact Beds in the United States. After the adoption of contact beds by Manchester in 1900, many plants of this type were built in the United States, particularly in the middle-western states. There are at present about twenty such plants in operation in the United States, of which six are in New Jersey and eight in Ohio. The most important contact installation along the Atlantic seaboard is at Plainfield, N. J., designed by

Mr. J. O. Osgood, with Mr. Andrew J. Gavett in charge of construction. The population of this city is about 20,000, and the sewage flow about 1,000,000 gallons a day. Intermittent filtration was at first adopted, but the beds used were of too fine grain and clogged badly. In 1900 a septic tank and double contact filter system was substituted, and in 1905 the plant was considerably enlarged to meet the increased sewage flow. The present installation includes four septic tanks and sixteen contact beds. Two of the septic tanks are 100 feet long and 50.5 feet wide, the others 200 feet long and 50 feet wide. The working depth is six feet and the total capacity about 1,350,000 gallons. The tanks are covered with wooden roofs and are well provided with baffles and scum-boards. The effluent flows over a long aerating weir on its way to the primary contact beds. The septic tanks are so connected that they can be run in parallel or combined in series, as desired. In spite of much experimentation, which has included the seeding of the tanks with old cesspool material and the systematic breaking up of the scum upon their surface, these tanks have never operated very satisfactorily. In 1908 thorough preliminary screening was introduced and the plan was tried of operating the several tanks intermittently, each pair being used for three or four days in succession in order to give time for the solution of the suspended solids. After 14 months of operation on this plan the accumulated sludge and scum (mostly scum) amounted to 3 cubic yards per million gallons of sewage treated (Fuller, 1909).

The contact beds are of concrete construction, each 92 by 106 feet in area and 5 feet deep. The eight secondary beds are 5.5 feet lower than the primary beds, but of the same dimensions and filled with the same material. On the floor of each bed fourteen lines of 4-inch horseshoe draintile were laid, converging to a controlling gate chamber at the center of each group of four filters. Over and between these tiles was placed a 6-inch layer of coarse broken stone upon which rested the main body of the filling material, 3.5 feet of $\frac{1}{4}$ to $1\frac{1}{2}$ inch stone, slag or clinkers. Over all was a 1-foot layer of coarse broken stone in which dis-

tributing lines of vitrified pipe were laid (Fig. 78). Of these the principal feed main of 12-inch size was cemented; while the laterals down to 3-inch were laid with open joints. The beds have clogged rather seriously, and it has been found necessary to open up trenches, remove some of the filling material and lay extra underdrains. It is hoped that the removal of all the filling material in the beds may be thus avoided.

The Plainfield plant is operated entirely by hand. Of the eight contact beds in each set, six are used in regular rotation and the other two held in reserve. The ordinary cycle is a 12-hour one, two hours each for filling, standing full and emptying, and 6 hours for rest. Of the results the report of the State Sewerage Commission (N. J., 1907) is as follows: "The effluent from the primaries is almost clear. Its odor is musty but not offensive. The final effluent is clear but not sparkling. Its odor, when an outlet gate is first opened, indicates the presence of considerable free ammonia. As the bed empties, this disappears and an earthy odor takes its place. The drainings are almost odorless. The effluent is an excellent double contact filtrate effluent easily assimilable by the stream which receives it."

The following analytical data are quoted by Fuller (1909) for the period from September, 1908, to March, 1909:

TABLE LXXII

RESULTS OF OPERATION OF CONTACT BEDS AT PLAINFIELD, N. J.

September, 1908–March, 1909. (Fuller, 1909.)

	Parts per million.			Putrescibility.	Bacteria per c.c.
	Suspended solids.	Oxygen consumed.	Nitrogen as nitrates and nitrites		
Screened sewage	117	86	....	......	2,370,000
Septic sewage	53	51	....	......	1,150,000
Primary effluent	19	22	.9	+	810,000
Secondary effluent	11	11	4.4	0	470,000

Numerous small contact beds have been built in the West, but many of them, through want of careful supervision, have not proved a success. The chief trouble has been caused by mis-

management and by dosing the beds with septic tank effluent which contained a large amount of suspended matter. The beds have thus become quickly clogged and unable to take the amount of sewage required. The effluent from these contact beds has, however, been generally nonputrescible.

One of the largest of the contact bed plants installed in the West was built by Snow and Barbour at Mansfield, Ohio. Mansfield is a manufacturing center with a population of about 25,000, and the dry-weather flow is in the neighborhood of 1,000,000 gallons a day. The sewage is passed through a grit chamber fitted with an inclined screen 10 feet by $5\frac{1}{2}$ feet in area, made of $\frac{3}{4}$-inch wrought iron bars, set $1\frac{1}{2}$ inches apart. Over the screen chamber the sewage passes to four septic tanks. These tanks are of stone masonry and of concrete, built underground and covered with turf, so that nothing is visible except a slight elevation of land. Each of the four tanks is 52 feet by 92 feet 3 inches in area, and 9 feet 6 inches high, the normal water line being 7 feet.

The contact beds are five in number and laid out in the form of a circle, of which each unit forms a sector, with a regulator house at the center. The total area is an acre and a quarter. The beds are principally in excavation, but partly in fill, and the walls are of the natural excavated clay. The filters are filled to a depth of 4 feet 9 inches with $\frac{1}{4}$ to $\frac{3}{4}$ inch crushed cinders. Each bed is very thoroughly underdrained with open-joint tile pipe. At the end of each of the seven main branch underdrains is a 4-inch iron ventilating pipe. Distribution on the surface of the beds is effected by branched wooden carriers with hinged joints at each change of section. The arrangement of these carriers, with a general view of the beds themselves, is shown in Fig. 82.

The dosing of the contact beds is regulated by an ingenious patented automatic device. The sewage flows over aerating steps and thence to a cylindrical chamber under the gatehouse at the center of the contact area. Radiating from this chamber are five 8-inch pipes leading to the five beds. The flow of sewage

is directed on to any one filter by a revolving gate, cylindrical in shape, concentric with the chamber and machine fitted to its interior. This gate has a single port, which may be brought by its revolution opposite the inlet of any one of the beds. Below the influent chamber is a similar cylindrical chamber which receives the effluent from the filters through radiating pipes at the sides and discharges it through a bottom opening. The movement of the revolving gates in the two chambers is controlled by a float, which intermittently rises and falls, and in so doing operates the cylinders through a train of gears. The float, in turn, is actuated by the rise of sewage in the bed which happens

Fig. 82. General View of Contact Beds at Mansfield, Ohio.

to be filling. The complete filling of the float chamber is adjusted to a one-fifth revolution of the cylindrical gates, and the opening of the outlet from a given filter is arranged to follow two-fifths of a revolution after the opening of its inlet. Filling and standing full occupy about $2\frac{1}{4}$ hours, and the empty period is five hours. This automatic apparatus appears to have operated with marked success. Figure 83 is from a photograph furnished by Mr. F. A. Barbour, which illustrates the working of a model prepared to show the general principles involved. When the float chamber fills, the inlet of bed *III* closes, the outlet of *II* opens and the sewage flow is diverted to *IV*. Between the cylinders are seen the cogs by which the motion is transmitted.

The septic tanks work very unsatisfactorily, the effluent some-

times containing more suspended matter than the sewage, and this has led to very serious clogging of the beds; yet the effluent from the contact beds has been found by the State Board of

FIG. 83. Model of Dosing Device for Contact Beds (courtesy of F. A. Barbour).

Health to be uniformly of good appearance and nonputrescible. Analytical data for a 24-hour test in May, 1907, are tabulated below:

TABLE LXXIII

EFFICIENCY OF SEPTIC TANKS AND CONTACT BEDS AT MANSFIELD, OHIO

Parts per million. (Kimberly, 1908.)

	Oxygen consumed.		Nitrogen as —					Suspended solids.			
			Organic								
	Total.	Dissolved.	Total.	Dissolved.	Free ammonia.	Nitrite.	Nitrate.	Total.	Volatile.	Dissolved oxygen.	Fats.
Sewage..........	37.0	25	11.5	7.6	7.8	.2	.4	74	55	2.3	43
Septic effluent....	32.0	......	12 2	....	9.4	.05	....	110	41	2.5	....
Contact effluent..	9.2	......	.8	....	3.6	.04	2.6	40	21	2.5	....

The original cost of the Mansfield plant included \$8000 for land, \$15,870 for the construction of the septic tank and \$19,850 for the construction of the contact filters. The cinders used for filling the beds cost 85 cents per cubic yard in place. The cost of operation was \$5644 in 1906 and \$5260 in 1907, amounting to 47 and 44 cents per person contributing sewage; but more than half of this cost is for pumping.

Dibdin's Slate Beds. Mr. W. J. Dibdin, the originator of the contact bed, has devised a modification of it which is so radical as to deserve particular discussion. This is the slate bed for the removal of suspended solids, — or multiple surface biological bed. A firm believer in the contact system, Mr. Dibdin was confronted by the loss of capacity due to progressive clogging and the consequent necessity for washing and renewal of the filtering material. He was opposed to the use of the septic tank or any anaerobic process; some means for treating the crude sewage in aerobic beds was what he sought. He reasoned that an increase in the air content of the contact bed would not only increase the liquid capacity of the bed, but also allow a sufficient aerobic action, so that the solid matter left in the bed would be much less in quantity and much less decomposable than the suspended matter in the sewage. After various experiments he decided that if a contact bed was filled with layers of slate slabs separated by one-fourth to one-half inch spaces, very much better results would be brought about, particularly if the slate were so laid that when necessary the solid matter deposited or remaining in the bed could be washed out with the slate *in situ*.

After twelve months' laboratory work an experimental bed was first constructed at Devizes in January, 1904, and this proved so successful that the process was adopted for the whole of the sewage of the town, the beds coming into operation on the 12th of September, 1905. Since then the system has been installed in some fifty different places, in many cases displacing septic tanks, chemical treatment processes, etc. It has received the sanction of the Local Government Board for Ireland for the treatment of three million gallons per diem at the Sydenham

outfall site of the main drainage system of Belfast, and it is now being installed for the War Office at the Cunagh Camp, Ireland. The general construction of a slate bed is indicated in Fig. 84.

FIG. 84. Filling Slate Beds at Devizes (courtesy of W. J. Dibdin).

In the Devizes experimental beds the initial working capacity was 82 per cent of the total cubic content of the bed. After 14 months this was reduced to 50 per cent; the opening of the outlet valve to its full extent, so as to permit the beds to empty with a rush, increased the value to 64 per cent, and the removal of some of the slates at one side so as to permit the slates to be flushed out with a hose, restored the original capacity of 82 per cent.

The plant at Devizes has recently been examined by Mr. A. C. Carter (1909) for the Royal Commission on Sewage Disposal, and the results of his examination may be summarized as follows. It is to be noted that his examination was made in the month of April, when the winter accumulation of solid matters is worked off at its most rapid rate.

The dry-weather flow of sewage is about 200,000 gallons per day, and the installation consists of 9 slate beds for preliminary treatment and 8 fine coke beds for final treatment. Seven of the slate beds are 45 feet by 68 feet by 4 feet deep for dealing with the dry-weather flow; 2 are 80 feet by 68 feet by 4 feet, for dealing with storm water. They are constructed of superimposed layers of slate separated by pieces of slate blocks 2 inches to 2½ inches thick. At the bottom of the bed is an open space of about 6 inches (all later beds have only a 2-inch space) and the slate is built up to an average height of 3 feet 6 inches. The beds are usually filled to a depth of three feet. The calculated capacity of the smaller beds is 60,000 U. S. gallons, and that of the larger beds 108,000 gallons each.

Crude unscreened sewage is run directly on to the beds, requiring three hours for filling when the flow is normal. The sewage then stands in the beds in a quiescent state for two hours, when they are emptied, this taking about three hours. When the outlet valve is opened, a considerable amount of black suspended matter comes away with the liquid. Average samples of the first ten minutes' flow gave 304 parts per million. In times of storm the sewage is passed through a detritus tank to catch road grit before running on the slates. Daily analyses of composite samples of the sewage and effluent were made for one week with the following results. The effluent samples did not contain any of the suspended matters which come away during the first ten minutes.

TABLE LXXIV

ANALYSES OF SEWAGE AND EFFLUENT AT DEVIZES

Parts per million. (Carter, 1909.)

	Number of samples.	Crude unscreened sewage.	Slate bed effluent.	Percentage reduction.
Suspended solids	7	446	149	66.6
Volatile matter in these solids	7	346	125	63.9
Solids by centrifuge (vols.)	7	2667	849	68.1
Nitrogen as free ammonia	5	63	70.3	....
Nitrogen as albuminoid ammonia	5	15.7	11.6	26.1
Organic nitrogen	5	35.4	24.9	29.6
Total nitrogen	7	109.9	91.9	16.4
Oxygen absorbed in 4 hours at 27° C. from N/8 permanganate	7	215	133	38.1
Chlorine	....	201	181	....

From results given in the above table it was calculated that the amount of dry solid matter in the sewage amounted to 145 tons per annum. The amount in the effluent was 48.5 tons, leaving 96.8 tons to be dealt with on the beds. The amount of dry solid matter discharged from the beds as humus, calculated from one day's experiment, equalled 41 tons per annum, leaving 55.8 tons either digested or retained by the beds. It should be taken into consideration, however, that the above results are calculated from the data obtained during only one week in April, and very different results might have been obtained if the observations had continued for a longer period or if a week in autumn had been taken rather than a week in spring.

The deposit on the slates in the bed was examined by opening the bed at one side down to the bottom. It averaged about ½ inch in thickness and was full of small red worms. No offensive odors could be detected, the smell being exactly that of a damp cellar.

Thus it will be seen that the characteristic feature of the slate bed is that it is a device for reducing solids in suspension. The decrease in the dissolved organic matters is less than in the ordinary contact bed, although sufficient to render the effluent suitable for treatment on secondary beds or direct discharge into tidal estuaries, or on to land. Its working capacity is double that of the ordinary clinker bed, and thus one-half of the construction cost of the tanks for the bed is saved. After nearly five years' work the beds show no signs of requiring cleaning, but continue to maintain a good working capacity without the depth of deposit increasing.

One of the most interesting points about the slate beds is the prominent part played by the higher forms of organisms. All observers note the immense numbers of worms, insects, crustacea and arachnida present in addition to protozoa and bacteria. Dibdin (1909) believes that the activity of the slate bed is in large degree a digestion of the solid matter carried out by these higher organisms; and the same thing is probably

true to a less extent of the trickling bed, which frequently contains enormous numbers of worms.

The Peculiar Sewage Problem at Belfast, Ireland. At Belfast the disposal of sewage is complicated by an unusual condition, which makes sand or trickling filters unsatisfactory; and the contact bed has offered just the necessary solution of the difficulty. As a rule, oxidation is the ultimate aim of sewage purification. At Belfast, however, the main condition which has made sewage purification urgent has been the heavy growth of seaweed in the harbor. The sea lettuce, *Ulva latissima*, and other Algæ flourish in such enormous quantity in Belfast Lough, that when the masses of seaweed are washed ashore and putrify they cause a nuisance of grave extent. A study of the distribution of the seaweed in relation to the pollution of the harbor made it clear that the excessive growths were due to the fertilizing action of the nitrogen of the sewage. Such green plants can utilize unoxidized nitrogen; but they can also utilize nitrogen in the form of nitrates and nitrites. So, even the complete nitrification of this sewage would not suffice. The actual removal of some considerable portion of the nitrogen seems to be the only way out of the difficulty.

Extensive investigations carried out by Dr. E. A. Letts have shown that treatment of crude Belfast sewage in double contact beds would remove 51 per cent of the free ammonia, and 71 per cent of albuminoid ammonia and oxygen consumed with but little formation of nitrates. Combined septic and double contact treatment gave better results, — 87 per cent purification measured by free ammonia, 73 per cent measured by albuminoid ammonia and 78 per cent measured by oxygen consumed. In this case, however, 6.2 parts per million of nitrogen as nitrates appeared in the effluent. Treatment by a septic tank and trickling filter gave still better purification with one-third the filter area; but the nitrate formation was naturally much greater than with the contact bed.

Trickling beds seemed therefore to offer the best method for purification, if the nitrates could somehow be removed. Dr. Letts, therefore, hit on the expedient of supplementing the

trickling process by secondary denitrifying beds on the contact plan. He found by mixing a nitrate solution with septic effluent that 25 parts of nitrogen per million disappeared in 24 hours. A small contact bed was constructed containing $1\frac{1}{2}$ feet of $\frac{1}{4}$-inch broken brick. This was dosed with mixtures of various proportions of septic effluent and trickling filter effluent giving the general results tabulated below.

TABLE LXXV

RESULTS OF SEPTIC, TRICKLING AND DENITRIFYING TREATMENT AT BELFAST

Parts per million. (Letts, 1908.)

	Septic effluent.	Trickling effluent.	Denitrified effluent.	Per cent purification.
Nitrogen as free ammonia	32.0	9.6	14.3	61
Nitrogen as albuminoid ammonia	6.8	3.4	3.7	46
Nitrogen as nitrates	....	16.6	1.7	....
Oxygen consumed	65.3	24.5	23.1	57

By using a larger proportion of trickling effluent the purification in organic constituents could be improved to over 80 per cent but a larger amount of nitrates would be left. The method finally recommended is as follows (Letts, 1908):

" After screening, and the removal of road detritus, the sewage should be submitted to a process of sedimentation in suitable tanks for a period of six hours, then a portion thus clarified should be treated on sprinkling filters, after which the resulting effluent should be mixed with the remainder of the tank effluent, and the mixture submitted to further treatment in a denitrifying bed, then discharged into ponds or lagoons, and thence into the Lough. Provisionally, a mixture of equal volumes of the tank and sprinkler effluents may be treated on the denitrifying beds, and the working cycle of the latter may be 4 hours' contact and 2 hours' rest; but it is quite possible that future experience may show that these conditions may be so modified as to induce a more complete purification than that hitherto obtained, which amounts to about 80 per cent in those factors which affect the growth of the *Ulva.*"

The main point of interest in the investigations is the light they throw on the special value of the contact system in the elimination, as distinguished from the mere oxidation, of nitrogen. The Belfast problem is a rare but not a unique one; and wherever the actual removal of nitrogen is for any reason desirable, the contact bed offers a peculiarly satisfactory method of treatment.

FIG. 85. Small Contact Installation in England.

Advantages and Disadvantages of Contact Treatment. The contact system is well established as one of the satisfactory methods of sewage disposal. Double contact will yield a stable effluent of sufficiently good character to be discharged into almost any stream. By comparison with the trickling filter, however, the contact bed is in general distinctly an inferior process from the

standpoint of cost efficiency. Comparative estimates for the two processes made by both English and American engineers will be quoted in the following chapter. The distributing system for the trickling bed is costly, both in construction and in operation; but the area required for trickling filters is from one-third to one-half that needed for contact beds, and the necessity for removing and washing contact material greatly increases the expense of operation in the latter case.

For certain special problems, however, the contact bed is better suited than the trickling filter. Its effluent differs from that of the trickling filter in two important respects. It contains only a small proportion of mineral nitrogen and is comparatively free from suspended solids. It will therefore meet the requirements of a peculiar case like that of Belfast; and it will often serve for plants where the disposal of the suspended solids in the trickling effluent might prove to be a burden. Another distinct advantage of the contact bed under certain conditions is the low head under which it can be operated. A trickling filter requires at the least 8 feet of head for the bed itself and for the distributing apparatus; while a double contact bed could, if necessary, be crowded into 5 feet. Altogether a contact installation lends itself to compact and inconspicuous construction, which is of much practical importance in the design of small plants for institutions or for private houses. The contact bed produces less odor than the trickling filter and does not breed flies as the trickling filter does. It may therefore safely be installed much nearer to dwellings. Another advantage in the contact system for small disposal plants lies in the fact that it adapts itself more readily to marked irregularities of flow than does the trickling bed. If, however, plants of this type are designed to work under the control of automatic apparatus, it must be remembered that a lack of careful supervision will mean certain and inevitable failure.

CHAPTER XI

PURIFICATION OF SEWAGE ON TRICKLING OR PERCOLATING BEDS

Historical Development of the Trickling or Percolating Filter. At about the same time that Dibdin was working out the principles of contact treatment, several engineers in England and America were laying the foundation for another method of purification by rapid filtration through coarse materials, on purely aerobic lines. As at Barking, the reduction of the filtration area necessary for sand filters was the chief end in view. Experiments at Lawrence on the filtration of sewage "through clean gravelstones larger than robins' eggs" had already furnished the first suggestion of such a process. In 1892 Hazen started a filter of one-fifth inch material which received four doses of sewage a day and was artificially aerated. The rate was increased from 140,000 at the start to 480,000 gallons per acre per day. The surface clogged badly, but the effluent was good, showing 30 parts per million of nitrates. This filter was dosed with an automatic siphon; but it was clear that in using coarse material some device must be introduced to secure a rather slow and regular passage of sewage through the bed. The first method adopted for this purpose was the spreading over the surface of a thin layer of finer material; but this closed the surface of the bed and required forced aeration for the maintenance of aerobic conditions. In 1892 Lowcock, at Malvern, England, constructed a gravel filter with a sand layer on its surface and filtered chemical effluent at a rate of nearly 300,000 gallons per acre per day, forcing air under pressure into the middle layer of the bed. A good effluent was obtained and the filter was operated for fifty-one days without rest (Lowcock, 1894). Similar filters were later constructed at Wolverhampton and at Tipton (Rideal,

1901). At both places ordinary trickling filters have since been installed. In the United States, Waring was attempting at the same time to use the principle of forced aeration. He obtained a patent on his process as early as 1891, and carried out a series of experiments at Newport in 1894 on "the mechanical straining out of all solid matters carried in suspension in sewage and their subsequent destruction by forced aeration and the purification of the clarified sewage by bacterial oxidation of its dissolved organic matter in an artificially aerated filter." The sewage, in Waring's words, "trickles down in a thin film over the surfaces of the particles of coke or other filtering material, while through the voids between the particles and in immediate contact with the trickling films of liquid a current of air is constantly rising, being introduced at the bottom of the tank by a blower." Fine gravel on the surface of the main body of filtering material effected the even distribution of the sewage, and the grosser suspended solids were removed by preliminary straining through broken stone. It is stated in the report of these experiments that the aerators removed "over 95 per cent of the organic nitrogen of a strainer effluent applied at a rate of at least 800,000 gallons per acre per day" (Waring, 1895).

Waring's principle of oxidation was undoubtedly correct; but the method of forced aeration is of more doubtful expediency. The difficulty of maintaining a sufficient supply of air by forced aeration is manifest, and the plants actually installed on the Waring plan have not generally operated with marked success. The best example of the process could be seen until recently at East Cleveland, Ohio. This plant included .112 acre of primary strainers, .056 acre of secondary strainers and .248 acre of aerator-beds and was designed for a sewage flow of 300,000 gallons a day. This rate was almost doubled, as the result of an increased number of connections; and under these severe conditions the beds, — strainers and aerators alike, — became badly clogged and the effluent was imperfectly purified. Extensions of the Cleveland sewerage system finally led to the abandonment of the plant.

A practically successful solution of the problem of rapid treatment under aerobic conditions was finally reached along another line, by abandoning the application of sewage in bulk, with artificial aeration of the filter from below, and resorting instead to the device of applying the sewage continuously, or at very frequent intervals, in a fine spray distributed evenly over the whole surface of the bed. By this means the too rapid flow of large currents of sewage is prevented, and at the same time air is drawn

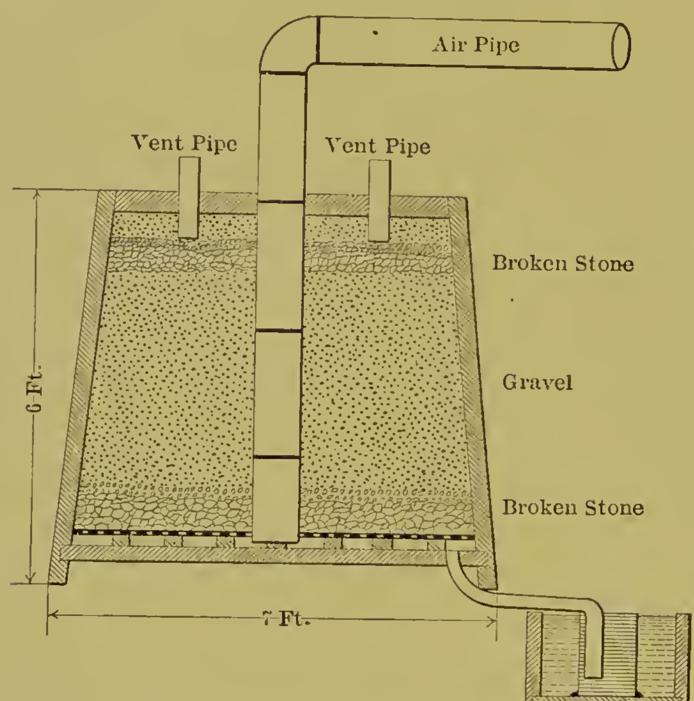


FIG. 86. Diagram of the Waring Filter.

in for oxidation from the top and sides of the filter. Sewage applied in this way trickles in thin films over the surface of the filling material, carrying with it atmospheric oxygen, so that the voids are continually filled with air, the oxygen content of which in practice does not become seriously exhausted. The air supply under the best conditions may amount to five times the volume of sewage. The material over which films of sew-

age continuously trickle supports an active growth of micro-organisms; and everything favors their maximum activity. The process is analogous to the cultivation of acetic-acid bacteria in vinegar manufacture by the flow of weak alcoholic liquor over shavings. The complications introduced by "a series of compensating errors of surfeiting and starvation" are exchanged for a simple and constant condition. Under the name of the trickling filter, the percolating filter, the "intermittent continuous" filter, the sprinkling filter, etc., this process has come nearer than any other to realizing the ideal conditions for rapid purification.

The first description of a method for sewage treatment based on the plan of trickling over coarse material with natural aeration was published by Stoddart in 1893. In the next year the same investigator exhibited a model at the Bristol meeting of the British Medical Association in which sewage and other liquids were discharged in drops over a filter of coarse chalk. A solution of ammonium sulphate containing 140 parts per million of nitrogen was almost perfectly nitrified at a rate of 11,600,000 gallons per acre per day. Sewage was completely nitrified at a rate of 1,200,000 gallons per acre per day and well purified at 5,800,000 (Dibdin, 1903). A working filter was constructed by Stoddart on this principle at Horfield in 1899. The same principle was independently worked out by Corbett, the borough engineer of Salford, in a series of experiments begun in 1893 under the inspiration of the work of the Massachusetts State Board of Health. He first used wooden troughs for distribution. Later he raised these troughs to a height of several feet above the filters, so that the sewage fell on the bed in a shower. Finally he sprayed the sewage over the surface from sprinkler nozzles such as are in use to-day. Corbett's work played an important part in the development of the details of construction of the trickling bed. He showed that aeration could be improved by laying a false bottom of half-pipes on the floor of the filter; and he studied the effect of dividing the bed into horizontal layers separated by an air space.

Besides Stoddart and Corbett, three other pioneers, Ducat, Scott-Moncrieff, and Whittaker, must be mentioned in any outline of the development of the trickling filter. Colonel Ducat strongly urged the importance of thorough aeration, building filters with open sides to attain that end, and maintained that the aerobic process alone was entirely competent for the treatment of crude sewage. He installed a small filter at Hendon in 1897. Scott-Moncrieff carried the process to a logical extreme in a series of experiments at Ashtead in 1898, described in Chapter IX. He believed that several different types of organisms were concerned in the purifying process and that their separate and successive cultivation under perfect aerobic conditions would give the most favorable results. It is not clear, however, that there is any such complex division of labor between various groups of nitrifiers as was postulated by Scott-Moncrieff. Corbett's experiments indicated that the introduction of horizontal air space was not of advantage and tray filters have only rarely been built in recent years. At the same time Scott-Moncrieff's results are interesting as an illustration of the progress and extent of the nitrifying process under such conditions.

The Principles of Purification in Coarse-grain Beds. Even less is known about the fundamental chemical and bacterial problems in the case of the trickling filter than in that of the contact bed. Nitrification and nitrosification play a part and are subject to the same laws which operate in intermittent filtration. The curves plotted in Fig. 87 bring out these two distinct processes in operation in the Technology experimental filters at Boston (Winslow and Phelps, 1907) and show the dependence of nitrification upon nitrosification and the dependence of nitrosification upon temperature. The filters started in October did not form mineral nitrogen in large amounts until the warm weather of the following summer. By September nitrite formation reached a maximum, and immediately afterwards the production of nitrates began in earnest, and, once started, proceeded successfully through the winter.

Sometimes, as in Scott-Moncrieff's experiments, the nitrifying processes play a predominant part. As a rule, however, the organic nitrogen is by no means entirely converted into nitrates. Frequently the total nitrogen and free ammonia values are only

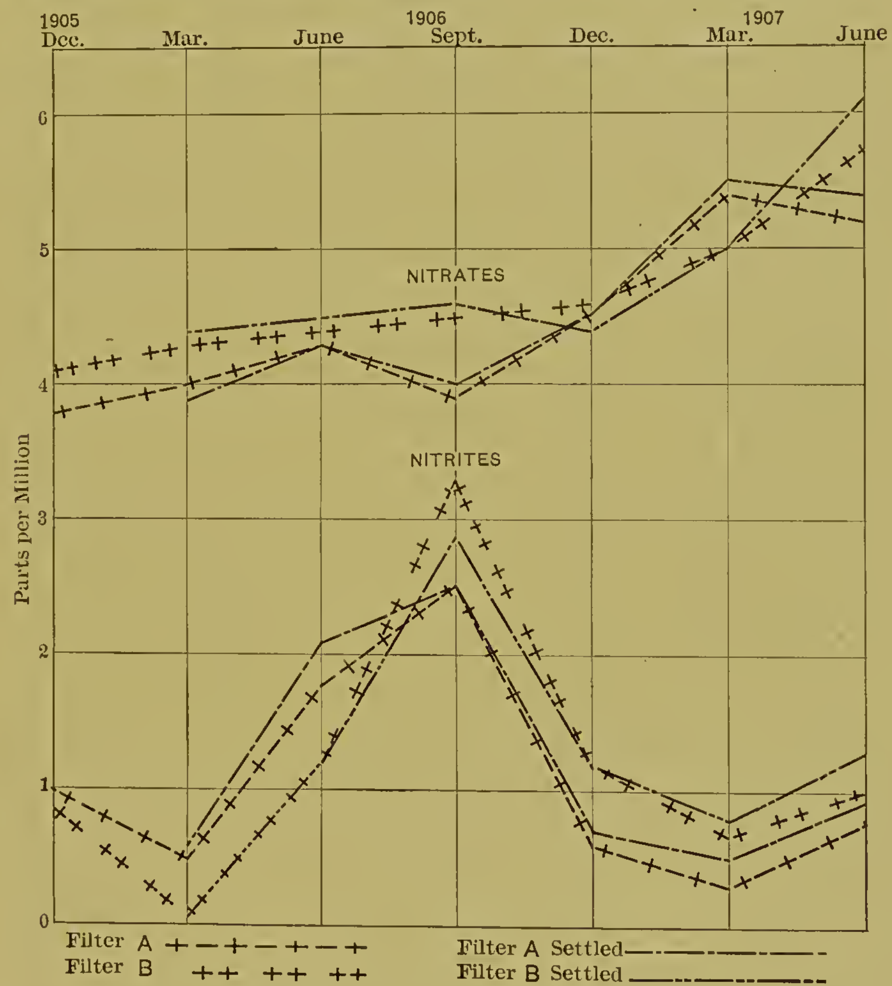


Fig. 87. Nitrification and Nitrosification in the Technology Trickling Filters.

reduced by 25–50 per cent; and yet the effluent may be stable, since it contains oxygen in sufficient amount to take care of a considerable proportion of decomposable organic matter. It is clear, from such results as those tabulated on page 320, that stability is often attained with an insignificant reduction of organic nitrogen.

TABLE LXXVI

RESULTS OF TRICKLING FILTRATION AT BOSTON, JANUARY–JUNE, 1907

Parts per million. (Winslow and Phelps, 1907.)

	Nitrogen as —					Putrescibility. Per cent samples stable for	
	Organic Nitrogen		Free ammonia.	Nitrites.	Nitrates.	2 days.	14 days
	Total.	Dissolved.					
Septic effluent........	4.7	2.5	18.7	0.0	0.0	......	......
Filter effluent........	3.4	1.4	12.0	.8	5.3	95.1	82.6

There is of course no doubt that the processes which go on in the trickling filter are in many ways similar to those which take place in the contact bed. Physical forces of adhesion and adsorption retain solid and liquid constituents on the surfaces of the filling material and the bacteria of the surface films subsequently effect the ultimate purifying changes. Both chemical and physical actions differ, however, in detail. Instead of decomposition into free nitrogen, the trickling bed forms nitrates and stable organic compounds; and the action of the trickling bed upon suspended solids is notably characteristic. The contact bed destroys a considerable part of the suspended solids applied to it; the trickling bed does not. The net amount of suspended solids discharged in the effluent is just about equal to that applied. It must be understood, however, that the material which the trickling bed discharges is by no means the same organic matter which it receives. It is in much denser particles, as indicated by the rapidity with which it may be removed by sedimentation. Under the microscope the various kinds of débris characteristic of sewage are absent, and instead there are in the main structureless flocculent masses, like the amorphous matter found in the microscopical examination of water. Furthermore, the filter has, in relation to suspended solids, a peculiar and definite annual cycle. During the autumn and winter months it stores suspended solids; and in the spring it discharges the accumulation. The history of this

phenomenon in the case of the Technology filters is shown in Fig. 88 (Winslow and Phelps, 1907). The cause of the spring break-up in the beds is uncertain. The low vitality of bacteria in winter and the increased growth with the warm weather of spring may both play a part. In any case, the delicate adjustment of a biological mechanism is here strikingly illustrated.

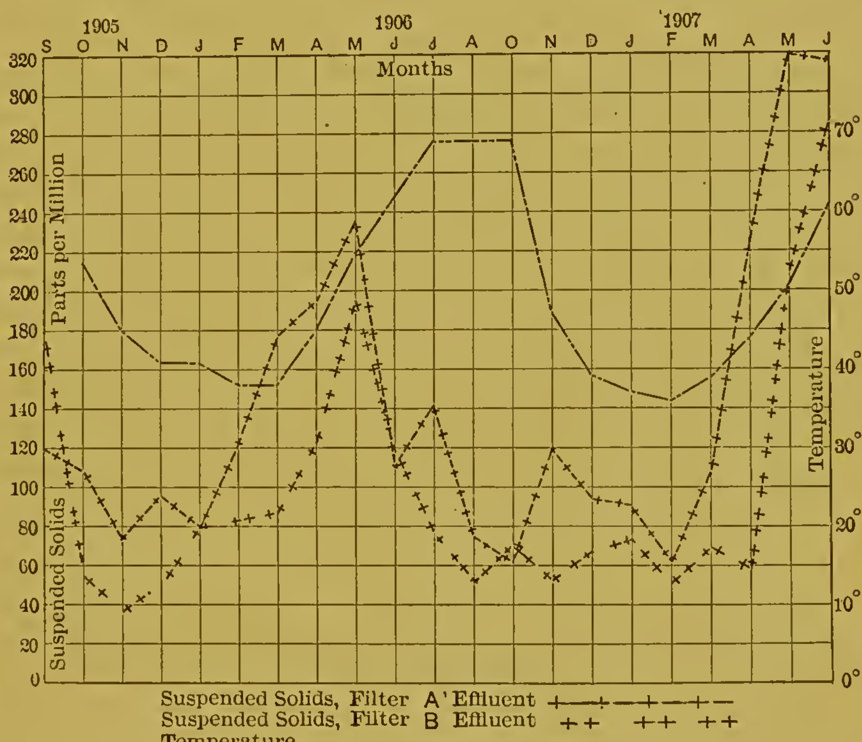


FIG. 88. Yearly Cycle of Suspended Solids, Technology Trickling Filters.

Mr. Rudolph Hering (1909) in a very suggestive summary of the principles of sewage treatment, has pointed out that the main engineering factors in the operation of the trickling bed are three in number,—the area of bacterial surface on the stones in the filter, the amount of oxygen available for the oxidation of the organic matter and the time of exposure. The temperature and the proportion of suspended matter in the sewage are classed as minor variables.

The bacterial surface will of course vary inversely with the size of the filling material, and also, according to many authorities, with the nature of its surface. Minor irregularities will soon be covered over, but Mr. Hering believes that the deeper films in the depressions will be active, nevertheless, through their absorptive power; and he calculates, assuming the surface of a glass sphere as 1, that the effective surface for gravel of the same average diameter would be 1.5, for broken stone 2 and for slag 2.5 or over. These conclusions are scarcely borne out by experimental results. At Hamburg, Salford and York, slag was better, but only slightly better, than smoother material; at Buxton and Tipton, on the other hand, coal proved superior to more porous substances. In any case, the size of the filling material is the chief controlling factor. Mr. Hering calculates that at Hanley $\frac{3}{8}$-inch saggers yield a bacterial surface of 135 square feet in every cubic foot of the bed; at Birmingham, $1\frac{3}{4}$-inch slag and granite give 60 square feet; and at Wilmersdorf 5-inch coke gives only 25 square feet. If we put aside Mr. Hering's correction for the roughness of the material, these figures become respectively 90, 34 and 10, an even more striking comparison.

The amount of air supply required is less perfectly known than any other factor. Rideal calculates that to convert the nitrogen of sewage into nitrates one-half gallon of air must be supplied for every gallon of English sewage. Hering estimates the air supply for the three plants cited above at from 4 to 7 cubic feet per day for the sewage of each person treated.

The time of passage through trickling beds varies with the rate of flow and with the size of the filling material; and Clifford (1908), in a very pretty study of this problem, has expressed the relation by a mathematical formula. He distinguishes two portions of liquid in a trickling bed at a given time,— the absorbed water, held in the pores of the material, and the interstitial water, on its surfaces. The absorbed water was determined by weighing the dry material, then submerging it for a considerable period, wiping with a cloth and weighing again. It amounted to 2.4 United States gallons per cubic yard of bed with gravel, and to

12.9 gallons with coke breeze. The interstitial water was found by subtracting the absorbed water from the total water held in the bed at a given time, when in actual operation. Interstitial water varied from 9.5 to 19.6 United States gallons per cubic yard, according to the fineness of the filling material, the rate of filtration being 240 United States gallons per square yard per day. An increase in rate of 60 gallons per square yard increased the interstitial water by .5 gallons.

The time of passage through a 2.4-foot bed was found by Clifford to vary directly with the amount of interstitial water (or, in other words, inversely with the size of the material), and inversely with the rate of application. He expressed the relation by the formula

$$cT = \frac{I}{R},$$

T being the time in minutes, I the interstitial water in gallons per cubic yard of bed, R the rate of application in gallons per square yard per hour, and c a constant. The average value deduced for c, with a bed 3 feet deep, was .03. The various depths of different materials necessary to secure a time of passage of 100 minutes are tabulated below:

TABLE LXXVII

DEPTH OF BED NECESSARY TO SECURE A 100-MINUTE PERIOD

(Clifford, 1908.)

Gravel.		Coal.		Slag.		Clinker.	
Grade.	Depth.	Grade.	Depth.	Grade.	Depth.	Grade.	Depth.
$1''-\frac{3}{4}''$	8′ 3″	$\frac{3}{4}''-\frac{5}{8}''$	7′ 2″	$1\frac{1}{4}''-\frac{3}{4}''$	6′ 1″	$1''-\frac{3}{4}''$	6′ 1″
$\frac{3}{4}''-\frac{5}{8}''$	7′ 3″	$\frac{1}{2}''-\frac{3}{8}''$	5′ 2″	$\frac{3}{4}''-\frac{5}{8}''$	5′ 4″	$\frac{5}{8}''-\frac{3}{8}''$	3′ 9″
$\frac{5}{8}''-\frac{1}{2}''$	6′ 7″	$\frac{3}{8}''-\frac{1}{4}''$	4′ 5″	$\frac{5}{8}''-\frac{3}{8}''$	4′ 9″	....	......
$\frac{1}{2}''-\frac{1}{4}''$	4′ 4″	$\frac{1}{4}''-\frac{1}{8}''$	4′ 0″	$\frac{3}{8}''-\frac{1}{4}''$	4′ 0″	....	......
....	....	....	....	$\frac{1}{4}''-\frac{1}{8}''$	3′ 10″	....	......

Mr. Hering has combined the three variables of bacterial surface, air supply and time in the following formula:

$$p = b\,a\,t,$$

where p = the degree of purification;
b = the area of bacterial film per person in square feet;
a = air supply necessary for the purification of the sewage of one person per day in cubic feet; and
t = the time of passage through the filter in minutes.

The application of this formula is illustrated by the following table:

TABLE LXXVIII

FUNDAMENTALS OF SEWAGE PURIFICATION

(Hering, 1909.)

City.	b Bacterial surface per person, sq. ft.	a Air supply per person, cu. ft. per day.	t Time of purification, minutes.	p Calculated degree of purification.	Actual character of effluent.
Hanley	710	4	76	216,000	Excellent
Birmingham	367	5	40	73,400	Fair
Wilmersdorf	160	7	15	16,800	Tolerable

The chief practical variables in the construction of trickling beds are the size of the material and the depth, the former affecting Hering's factor, b, and both affecting t. The formula is very suggestive in its emphasis on the value of these two factors. Perhaps some day accumulated experience will make it possible to fix a value for p by which we can determine for any given sewage the depth of a bed which will yield a stable effluent with a given material, or conversely the largest material which will suffice for purification with a given depth.

Construction of Trickling Beds. Reduced to its simplest terms, a trickling filter is a heap of selected filling material; and some of the earlier filters, at Bristol, for example, were little more than this in actual fact (see Fig. 89). Sometimes local conditions of head make it necessary to build the beds below the surface of the ground; in this case it has been thought economical to form the sides of the excavation to their natural angle of repose. In some of these cases the filter proper is surrounded by a battered wall of filtering material. This, however, is not an economical method. At Birmingham it was found that building a rubble wall around

an acre filter cost only half as much as the extra labor and material required for a battered wall of filtering material. As a general rule, therefore, it will be found best to build trickling filters with structural walls of some sort. Raikes (1908) recommends for filters below the surface that the thickness of the wall should be at least 14 inches at the top and not less than one-quarter the height of the

FIG. 89. View of Trickling Filter at Bristol (courtesy of F. Wallis Stoddart).

wall at the base; for filters above ground the diameter should be 14 inches at the top and should increase $4\frac{1}{2}$ inches for every yard of depth. The floor of the filter must in any case be smooth and impervious, so as to insure prompt and efficient drainage; cement concrete is generally the most suitable material.

The general form of the individual beds, whether circular, rectangular, triangular or hexagonal, as well as their relation to each other, will be largely determined by the form of distributing device

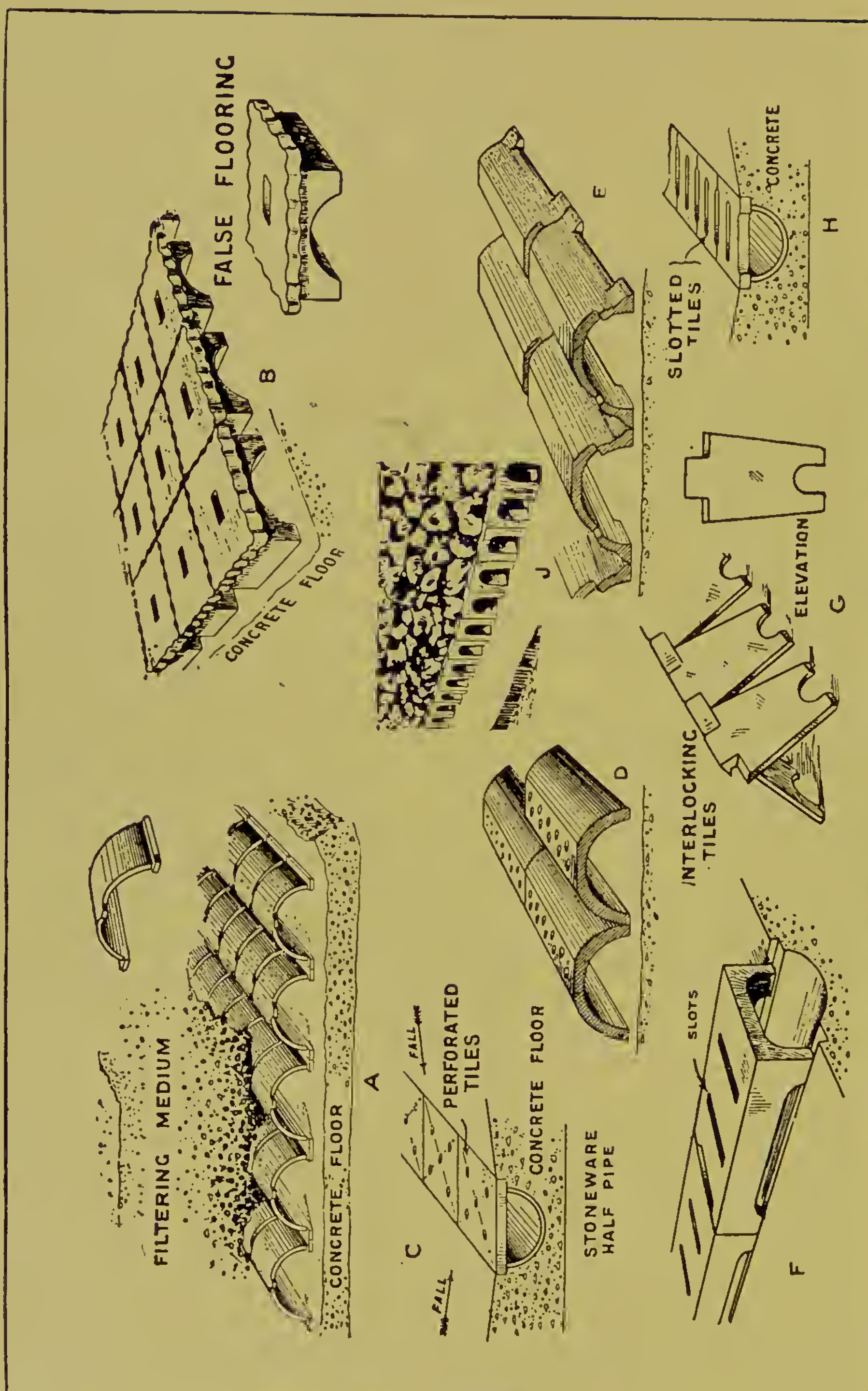


FIG. 90. Types of Underdrains for Trickling Beds (copied by permission from *Sewage Disposal Works* by H. P. Raikes, Constable Co., London).

adopted. Movable distributors require circular or rectangular beds or groups of hexagons; fixed sprinklers allow the filters to be laid out in large continuous areas of any indicated form.

Careful underdrainage is one of the most important essentials in the trickling filter, not only for the removal of the liquid with its load of suspended solids, but also for the satisfactory aeration of the bed. Sometimes the branch underdrains are laid at intervals; it is perhaps better to construct what is practically a false floor by the use of some of the devices shown in Fig. 90. The underdrains used at Columbus, Ohio, are of 6-inch half-tile laid in the manner indicated in Fig. 110. Main collectors should be of ample dimensions, and if possible so exposed that the wind may have free access to them. Those designed for the Columbus plant are concrete drains 24 and 36 inches in diameter. It is well to provide that underdrains should converge toward the center of the filter for economy of head and where possible they should pass out through the walls at their upper ends to allow for flushing. Taylor (1909 *b*) has designed an elaborate and ingenious false floor for the Waterbury filters and has provided for aeration by special ventilation chimneys equipped with orienting cowls for utilizing the force of the wind.

Special facilities for aeration are provided in many English filters by open construction of the walls. The walls of the Ducat filter were built of open drainpipe inclined upward and connected with aerating drains at intervals in the body of the bed. The Whittaker-Bryant filters at Accrington and elsewhere are octagonal in shape, with walls of open brick and central open brick work aerating wells (Fig. 91). This sort of construction is probably unnecessary if free underdrainage is provided; but Mr. Hering (1909) believes that the open-side filters in England and the special air pipes at Wilmersdorf have proved of distinct advantage, and holds that the resting of the filters which has been found necessary at Birmingham might have been avoided by similar provisions. A considerable amount of air must certainly be supplied to the interior of the filter from some source. Dr. Fowler has calculated that the sewage going on at the top, even

when completely saturated with air, contains only one-fifteenth of the oxygen necessary for purification.

As in the case of contact beds, evidence in regard to the relative value of various kinds of filling material is somewhat conflicting. At Salford, slag was found somewhat better than polarite, gravel, coke, or clay. At York a well-controlled series of investigations

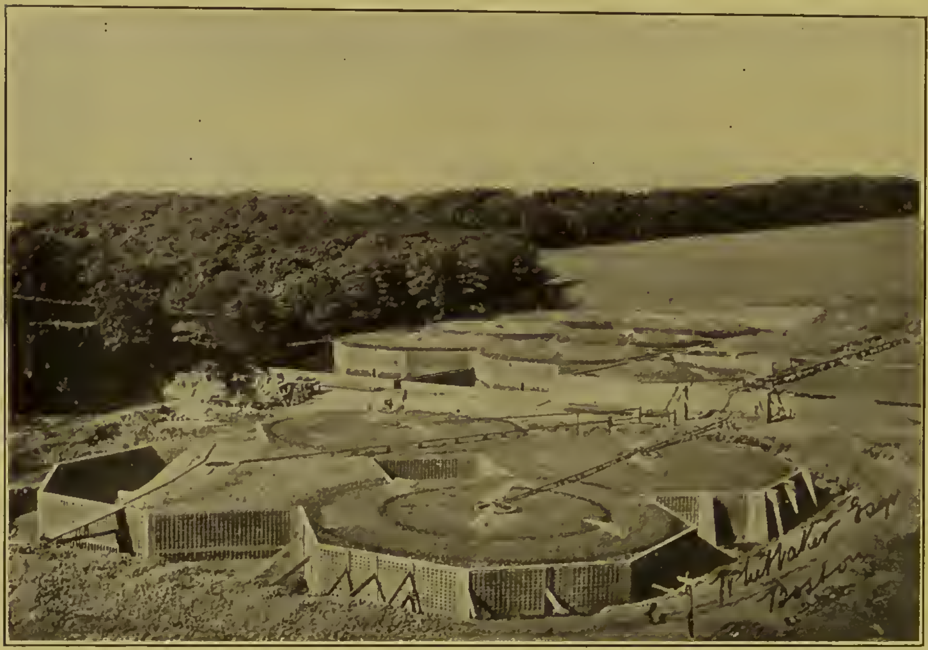


FIG. 91. Whittaker-Bryant Filter at Accrington (courtesy of Mr. Whittaker).

indicated, as shown in the table below, that coke and boiler slag (clinker) are slightly better than brick and blast-furnace slag.

TABLE LXXIX

EFFICIENCY OF TRICKLING FILTERS OF DIFFERENT MATERIAL AT YORK, ENGLAND

Parts per million. (Bredtschneider and Thumm, 1904.)

	Nitrogen as —		Oxygen consumed in 4 hours at 80° F.
	Albuminoid ammonia.	Nitrates.	
Raw sewage	13.9	0.0	82.9
Broken-brick effluent	1.4	18.4	10.0
Blast-furnace slag effluent	1.2	18.8	9.6
Coke effluent	.9	23.0	7.1
Boiler-slag (clinker) effluent	1.0	22.0	6.9

Coal has been found especially favorable to the process. At Buxton the effluents from destructor breeze and coke showed, respectively, 0.8 and 0.9 parts per million of albuminoid ammonia and 0.8 and 0.7 parts of nitrates, while a coal filter yielded only 0.4 part of albuminoid ammonia and 3.4 parts of nitrates (Barwise, 1904). Calmette (1909) has recently reported excellent results from experimental filters filled with fragments of peat; but

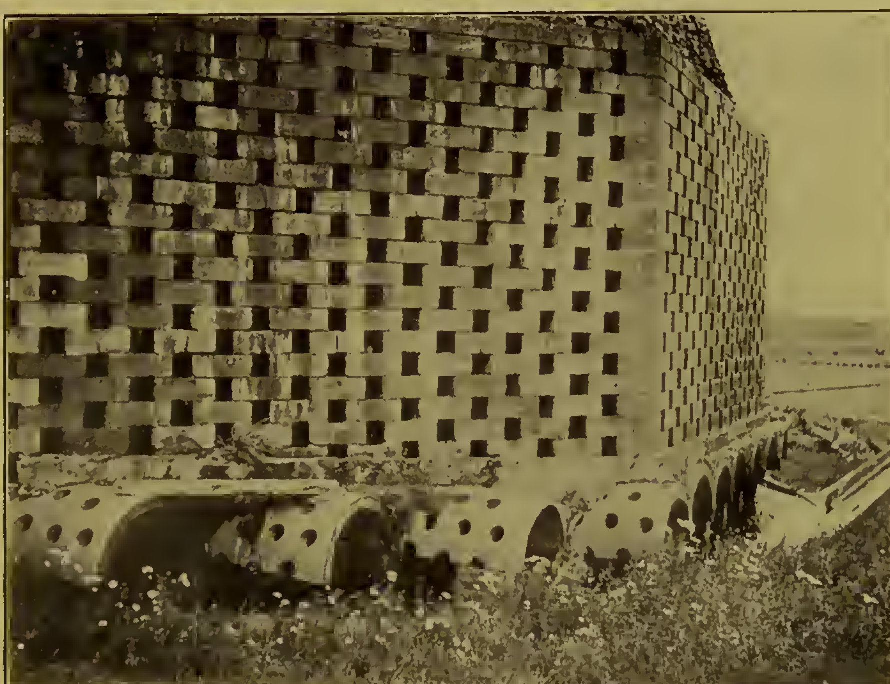


Fig. 92. Open Brickwork Construction of Trickling Filter at Leeds.

he found that cinders mixed with carbonate of lime gave the best results of all.

On the whole, it seems probable that any hard material will serve well for the trickling filter. Raikes (1908), in reviewing English experience with various materials, mentions particularly burnt clay, coal, coke, gravel, slag, sandstone, granite, furnace clinker, overburnt bricks and saggers. Burnt clay has worked badly, soon disintegrating into powder. Coal may serve, if a

hard variety can be cheaply obtained, but it is more liable to break down than still harder materials. Coke settles considerably and forms a good deal of wasteful dust when put in place. Sandstone takes a long time to drain and forms much dust when it is broken. Raikes holds that gravel is too smooth for supporting the most luxuriant bacterial films. Slag and furnace clinker are satisfactory if of good quality, but the quality varies greatly, depending upon the source of the material. Some clinkers are soft and friable, and slag containing lime or sulphur is apt to disintegrate. Overburnt brick, saggers and granite are hard and impervious and form excellent filling material where they are available.

The size of the filtering material to be used will be largely controlled by the character of the sewage treated, and particularly by the amount of suspended solids it contains. Considerable latitude may be allowed in this regard without seriously affecting purification. The elaborate experiments carried out by Reid at Hanley, cited in the table below, indicated that fragments from $\frac{3}{16}$ inch up to $1\frac{1}{2}$ inches yielded almost identical results (Hanley, 1904).

TABLE LXXX

EFFICIENCY OF TRICKLING FILTERS WITH MATERIAL OF VARIOUS SIZES AT HANLEY, ENGLAND

(Hanley, 1904; Wilcox and Reid, 1904.)

	Size of material (inches).	Analyses (parts per million).					
		Solids.		Nitrogen as —			Oxygen consumed in 4 hours at 80° F.
		Dissolved.	Suspended.	Free ammonia.	Organic.	Nitrates.	
Sewage	....	1,250	629	17.3	6.3	0	38.5
Septic effluent	....	1,050	44	15.0	2.2	0	17.3
Rectangular bed:							
Section 1	$\frac{3}{16}$ to $\frac{1}{8}$	1,120	4	.7	.2	17.5	2.7
Section 2	$\frac{1}{2}$ to $\frac{1}{8}$	1,120	3	.8	.3	17.3	2.8
Circular bed:							
Section 1	$\frac{3}{16}$ to $\frac{1}{8}$	1,120	2	.8	.2	16.6	2.4
Section 2	$\frac{1}{2}$ to $\frac{1}{8}$	1,130	14	.3	.2	15.3	2.6
Section 3	$\frac{1}{2}$ to $\frac{1}{4}$	1,130	7	.3	.2	16.2	2.5
Section 4	$1\frac{1}{2}$ to $\frac{1}{2}$	1,130	17	1.0	.4	16.2	3.3

With still larger material, purification might not be satisfactory, but a corresponding increase in depth could be made to compensate for the lessened area of bacterial surface. Under ordinary conditions we are working with a considerable factor of safety. Mr. Hering's calculations, cited on page 324, show that the Wilmersdorf beds of 5-inch coke with a bacterial surface of only 160 square feet per person give a tolerable effluent, while ordinary filters, like those at Birmingham, of $1\frac{3}{4}$ inch slag have twice as large a surface, and the fine beds at Hanley ($\frac{3}{8}$ inch saggers) have over four times as much. Barwise (1904) suggests $\frac{1}{8}$ to $\frac{1}{2}$ inch material, and Raikes (1908) places the limits at $\frac{1}{4}$ to $\frac{3}{4}$ inch. Among the witnesses before the Royal Sewage Commission, Garfield recommended $\frac{1}{16}$ to $\frac{3}{16}$ inch, Ducat $\frac{1}{8}$ to $\frac{1}{2}$ inch, Corbett $\frac{3}{16}$ to $\frac{3}{4}$ inch, Candy $\frac{3}{16}$ to $\frac{1}{2}$ inch for fine and $\frac{3}{4}$ inch to 3 inches for coarse beds, Harding $\frac{1}{4}$ inch to $1\frac{1}{2}$ inches for fine and over 3 inches for coarse beds, Whittaker 1 inch to $1\frac{1}{2}$ inches, and Stoddart 2 to 3 inches. In the United States, with weaker sewages and less satisfactory preparatory treatment, coarser beds are recommended. At Reading $1\frac{1}{2}$ to 4 inch blast furnace slag is used; and at Columbus, $1\frac{1}{4}$ to 4 inch limestone. At Boston the Technology experiments have indicated that crude sewage can be treated successfully in beds of $1\frac{1}{2}$ inch gravel, while a $\frac{1}{2}$ inch bed has clogged and pooled badly.

The depth of the trickling bed is controlled, on the one hand, by the strength of the sewage to be treated, and on the other hand by the size of the filtering material. With material between $\frac{1}{8}$ and $\frac{1}{4}$ of an inch, the sewage travels downward at a rate of from $\frac{1}{2}$ inch to 1 inch per minute, while with $\frac{1}{2}$ to $\frac{3}{4}$ inch material this rate is nearly doubled. With quite fine material a shallow filter will serve, and there may be no particular gain in deepening it. Thus, Table LXXXI, from the Hanley experiments, shows that a bed of $\frac{1}{8}$ inch to $\frac{1}{4}$ inch material carefully dosed by a moving distributor effected practically complete purification in a depth of only two feet, and excellent results even in one foot:

TABLE LXXXI

PURIFICATION IN TRICKLING BEDS OF VARIOUS DEPTHS AT HANLEY

Parts per million. (Raikes, 1908.)

	Suspended solids.		Free ammonia.	Alb. ammonia.	Oxygen consumed, 3 min.	Nitric nitrogen.
	Total.	Volatile.				
Sewage.........	635.0	285.0	21.54	9.72	18.62	.2
Detritus tank...	170.0	68.0	16.43	4.86	9.75	.2
Septic tank.....	76.0	38.0	17.16	3.40	8.36	.0
Filter, 1 foot....	2.5	1.6	.36	.52	.93	6.4
Filter, 2 feet....	.9	.5	.20	.37	.77	18.2
Filter, 3 feet....	1.4	.6	.09	.31	.60	17.5
Filter, 4.5 feet...	....	....	.43	.27	.70	17.0

Practically, four feet should probably be fixed as a minimum, even for a fine bed, to avoid the danger that streams of unpurified sewage may pass through channels formed by irregular packing of the material. With coarser filling the depth may have to be increased in proportion to the coarseness of the material, the strength of the sewage and the rate of filtration. Results obtained at Leeds with three successive filters of coarse coke made it clear that $3\frac{1}{2}$ feet was insufficient for adequate purification.

TABLE LXXXII

EFFICIENCY OF TRICKLING FILTERS AT LEEDS, ENGLAND

Parts per million. (Dibdin, 1903).

	Depth, feet.	Total solids.	Suspended solids.	Nitrogen as —		Oxygen consumed in 4 hours at 80° F.
				Free ammonia.	Albuminoid ammonia.	
Sewage.........	....	1,760	631	27.6	12.2	127.0
Effluent No. 1...	3.5	1,250	275	18.5	7.0	62.5
Effluent No. 2...	6.0	1,060	113	13.3	5.0	39.6
Effluent No. 3...	8.5	1,010	110	9.7	3.5	27.6

At Reading and Columbus, with $1\frac{1}{4}$ to 4 inch material, the filters have been designed 5 and $5\frac{1}{2}$ feet deep respectively. At Wilmersdorf, with 5-inch coke filling, even 8-foot beds do not yield a more than tolerable effluent. If filters deeper than four feet are undesirable on account of insufficient head, the same

end may be attained by increasing the area and decreasing the rate of filtration. Experiments of the British Royal Commission (R. S. C., 1908) at Horfield and Ilford led to the conclusion that "within somewhat wide limits of depth, and given ample aeration and good distribution, the same amount of work can be got out of a cubic yard of coarse material whether it is arranged in the form of a deep or of a shallow percolating filter." Careful comparison of two filters at Ilford, one 3 feet and the other 6 feet deep, showed slightly better results for the deep filter — 74.6 per cent purification in oxygen-consumed value against 71.8 per cent for the shallow bed.

The tables published in the final report of the Royal Sewage Commission give a good general idea of English practice in the construction and operation of trickling beds.

The Distribution of Sewage on Trickling Beds. The distribution of the sewage in a fine spray over its surface is the chief problem in the construction of a trickling bed and in its successful operation. The attempts of Lowcock and Waring to secure distribution by spreading a layer of fine material over the whole surface of the bed have been already described. This method is very rarely attempted, however, at the present day. Scott-Moncrieff and Ducat originally used tipping buckets and troughs placed at intervals over the filter, relying on the dash to distribute over intermediate areas. This plan has been tried experimentally at Hendon and Leeds. The distribution is of course imperfect, so that deep beds of rather coarse material should be used to insure purification without serious ponding. The apparatus requires pretty constant supervision to keep it in order; but the method has the advantage of adjusting itself easily to variations of flow and for small plants may prove satisfactory.

A third distinct type of distributor is the dripping tray, devised by F. Wallis Stoddart for use at Horfield. It is practically a series of channels, over the sides of which the sewage overflows continuously, dripping from a series of points on the underside, 360 points being allowed to a square yard. Theoretically it should secure a very even distribution. The channels are liable to

buckle, however, and it is difficult to keep them level. Furthermore, they are subject to serious clogging from fungous growths (Barwise, 1904).

Neither of these methods of distribution has been widely used. In England two systems have attained rather general acceptance, one involving the use of movable sprinklers which rotate or travel back and forth over the surface of the bed, driven by power or by the head of the sewage, and the other depending on the spraying of the sewage in jets from the orifices of a fixed system of piping. The first type, including all the various forms of movable distributors, is still in most general use in England.

Fig. 93. Trickling Beds Equipped with Candy-Whittaker Distributor (copied by permission from Barwise, 1904).

Movable distributors may again be divided into two classes, according as they are driven by the head of the sewage itself or actuated by special applications of power. The first to come into use were of the former type, and were generally operated on the principle of the Barker's Mill. They consisted of a number of perforated iron pipes radiating from a central pillar and revolving about it, receiving sewage from the center and discharging through holes all on one side of the pipes, so that the reaction

of the escaping jets caused the arms to revolve in the opposite direction. A view of the Candy-Whittaker distributor in Fig. 93 shows the general appearance of this form. In small plants the sewage is supplied to the arms from overhead by an annular trough revolving on a ring of ball bearings around the central supply pipe. With larger filters and longer arms, requiring a greater head of sewage, the construction of these troughs is difficult; and in any case stoppages in the revolving arms would lead to serious overflows at the center. In most cases, therefore, the arms are fed from below. This means that the apparatus must be water-tight under pressure; and this end is difficult to attain without introducing so much friction as to waste a considerable proportion of the available head. A number of ingenious methods of meeting this difficulty have been devised, of which one is illustrated in Fig. 94. The ball bearings on which the distributor revolves are in this case protected by a mercury seal. In another apparatus the weight of the distributor is borne on ball bearings placed at the top of the central pillar, while the bottom contact is made by a pair of smooth brass rings pressed together by the weight of the sewage but sliding freely upon each other. In revolving pipe sprinklers of large size, guy ropes must be provided for the support of the arms. In all of them, too, it is necessary to provide some means for stopping the discharge of sewage when the flow is too small to operate the apparatus properly. This is generally accomplished by the use of preliminary storage and siphon tanks, which operate the sprinklers intermittently at times of low flow.

Another form of revolving sprinkler designed by Mather and Platt is driven by a turbine wheel in the central pillar. Still a third entirely different type of automatic distributor depends on power developed by discharging the sewage over a sort of movable water wheel so that the impact of the sewage from the trough revolves the wheel. A distributor of this general plan (the Fiddian) is shown in Fig. 95, as adapted to a rectangular bed. The feed pipe is a siphon which moves in a trough at the side of the bed. Alternate sections of the discharge buckets empty in opposite

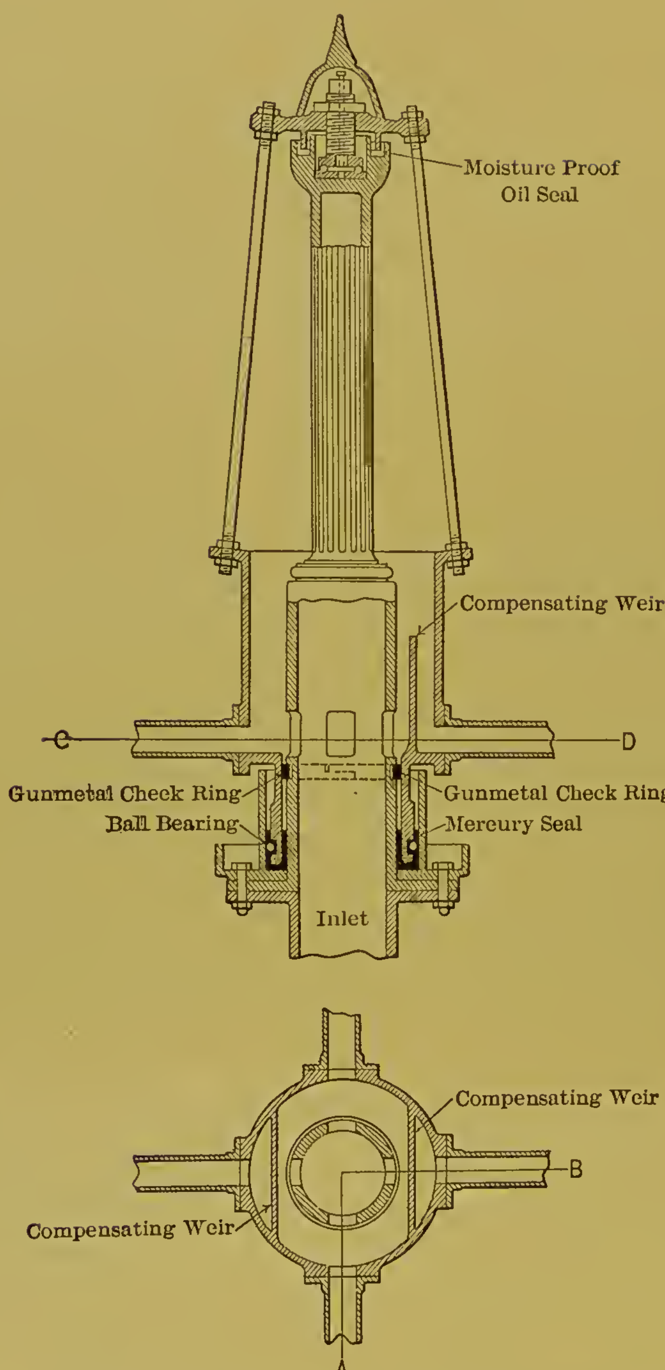


FIG. 94. Center Pillar for Mercury Seal Distributor (courtesy of The Patent Automatic Sewage Distributor, Limited).

directions, and at the end of each excursion a lever collides with a buffer and deflects the sewage to the other set of buckets, which drive the apparatus back again to the end of the bed from which it started. These Fiddian distributors are costly and require more power to drive them than do the revolving pipes. Their freedom from small openings liable to clogging, their adaptation to rectangular beds and the fact that they will operate under widely varying heads, are strong points in their favor.

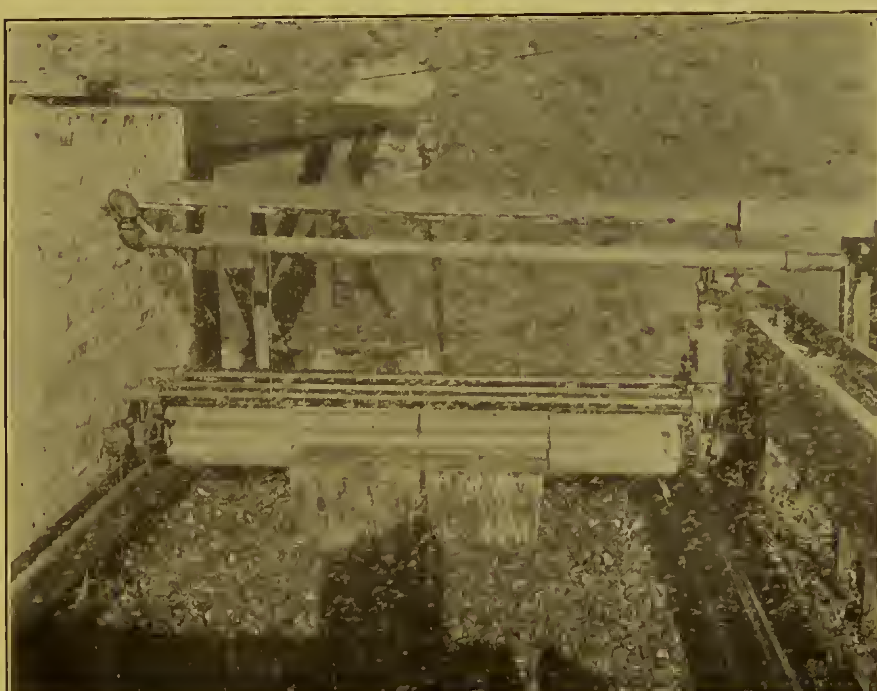


FIG. 95. Fiddian Distributor for Rectangular Bed, Technology Experiment Station, Boston.

Where there is insufficient head for any of these automatic devices, power-driven sprinklers have been introduced in many English towns. One of the earliest of these was devised by Mr. Scott-Moncrieff and is shown in Fig. 96 as it was used in experiments at Birmingham. It consists of a large open trough revolving about a vertical supply pipe at its center and supported on a circular rail at the periphery. The sewage overflows from a small

side trough divided into sections, to which it is admitted from the main trough in amounts proportional to the circular path covered by each section. The propelling power is furnished by an oil engine carried on the outer end of the trough. The distribution effected is excellent, but the weight of the apparatus is great and the cost of operation very high. At Hanley the distributor for a quarter-acre bed weighed 12 tons; a 45-lb. bridge rail around the filter was worn out in two years and a half; and the cost of

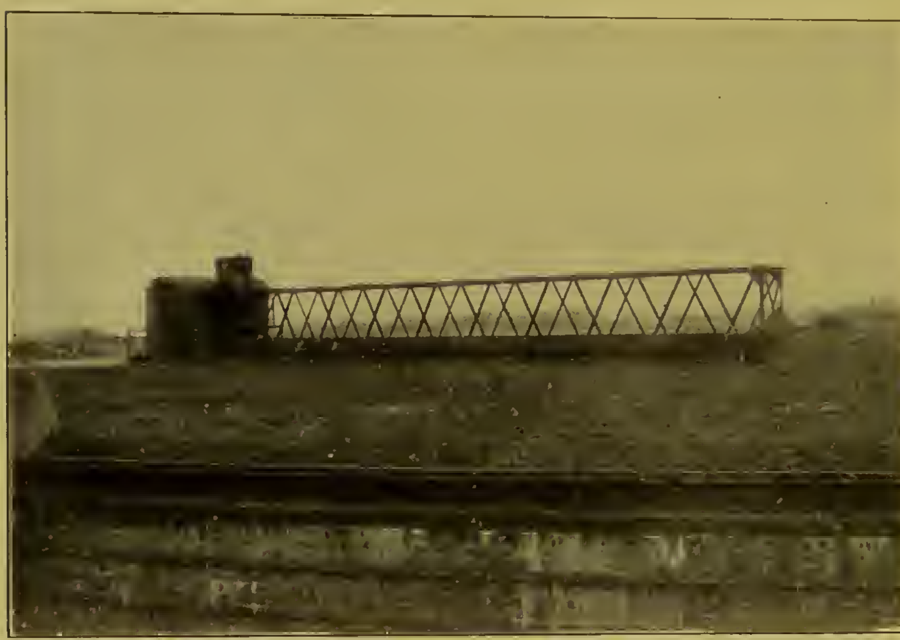


FIG. 96. Scott-Moncrieff's Revolving Distributor at Birmingham.

renewal and supervision amounted to ten dollars per million gallons of sewage distributed. In the method finally adopted at Hanley, the distributor itself is a perforated iron pipe moved back and forth on wheels by means of a wire rope, driven from an electric motor provided with automatic gear by which the motion is reversed at the end of each excursion.

All these various movable distributors may give excellent results as far as even distribution is concerned. They are costly, however, to install; they are liable to be frequently out of order under the best conditions; and they could not possibly be used in a

severe winter climate without protecting the whole filtering area by a roof. For these reasons they have never found favor in the United States; and there appear signs of reaction against them, even in England. At Birmingham Mr. Watson (1907) reports that the cost of sprinkler nozzles was only $2500 per acre against $5000 to $20,000 for movable distributors; and that the movable distributors were out of order for 8–28 per cent of the time.

The method of dosing trickling beds from fixed sprinkler nozzles was first, perhaps, developed at Salford, after various other attempts, with troughs and with a thin layer of sand laid over the surface of the main filter. Disc-like caps were placed over the openings of the pipes in some early experiments, in order to secure a good spray. Then the attempt was made to get a spray by the impact of two converging flows, and finally a special form of opening was designed to give a rotating movement to the stream. Two of the Salford nozzles are shown in Fig. 97. At Chesterfield and other towns in Derbyshire metal plates were placed over the orifice to break the sewage into fine spray. Most of the later types of nozzles attain this end by a cone or plug placed directly over the orifice and supported either by a central rod or by lateral arms. The Birmingham nozzle, shown in Fig. 97, has a $\frac{3}{8}$-inch opening through which passes a $\frac{7}{32}$-inch shank supporting a plug, which breaks the sewage up very effectually. Nozzles with such small openings clog badly, and at Birmingham it requires the constant attention of one man to every one and a half acres to keep them clear. In the United States larger orifices are therefore preferred, like the Columbus sprinkler shown in Fig. 97, which has a clear aperture of $\frac{9}{16}$ of an inch with an inverted cone supported above by lateral arms. The general character of the spray formed by these sharply contrasted sprinklers is indicated in Figs. 98 and 99; and the Columbus plant in actual operation is shown in Fig. 111. The spacing of the orifices and the head under which they must be operated naturally vary widely, according to the type of nozzle used. Taylor (1909 *c*) has recently published detailed studies of distribution from fixed sprinkler nozzles, in-

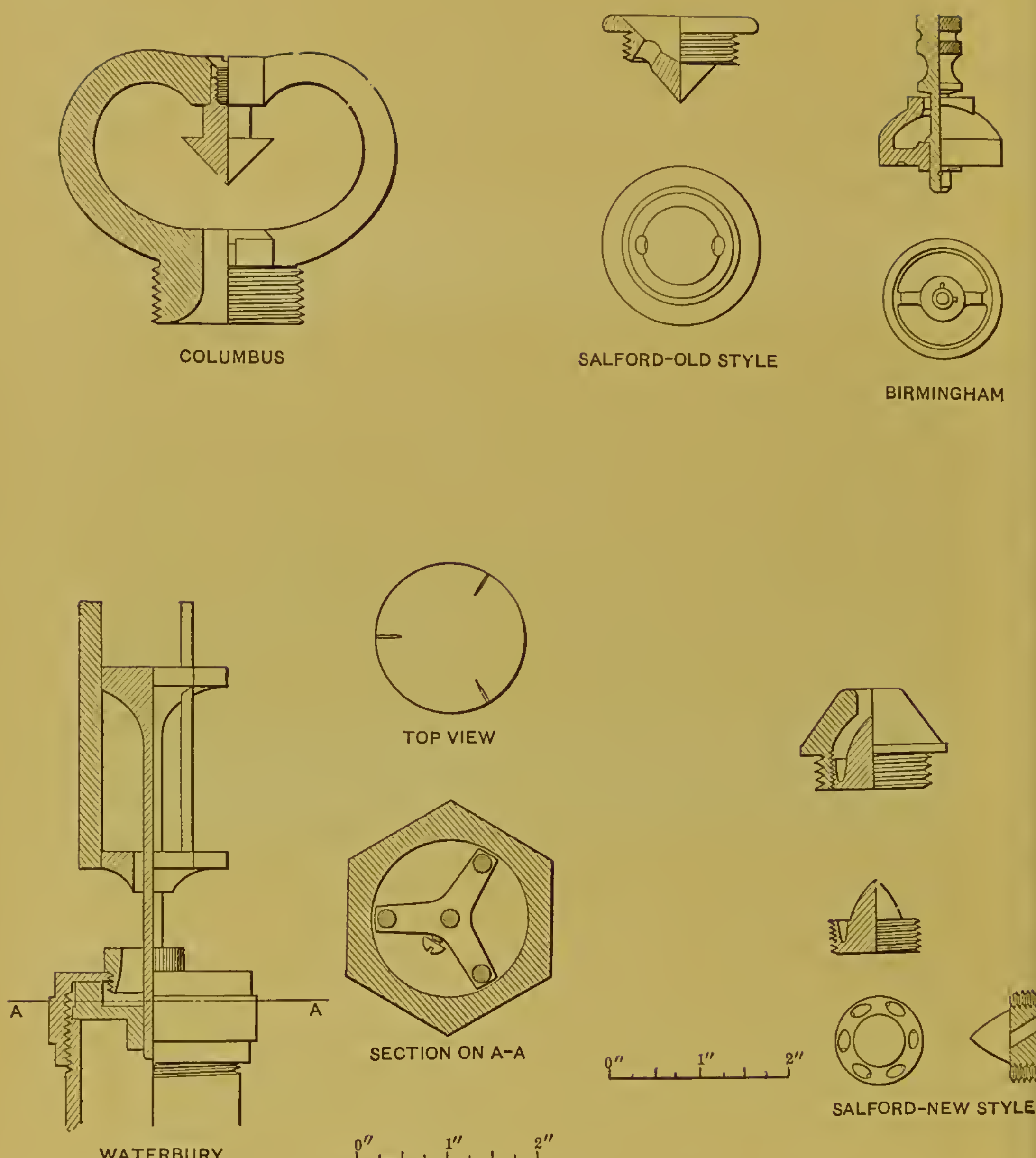


Fig. 97. Types of Nozzles for Fixed Sewage Sprinklers (Winslow, Phelps, Story and McRae, 1907).

FIG. 98. Form of Spray from Birmingham Nozzle.

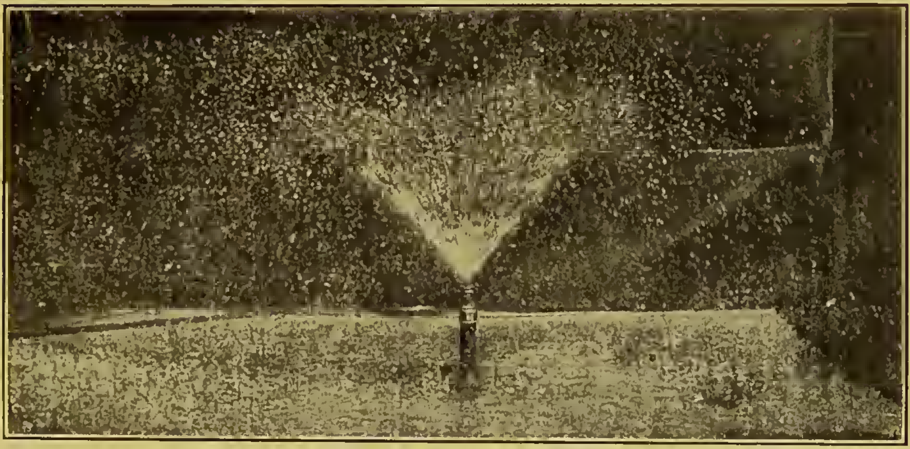


FIG. 99. Form of Spray from Columbus Nozzle.

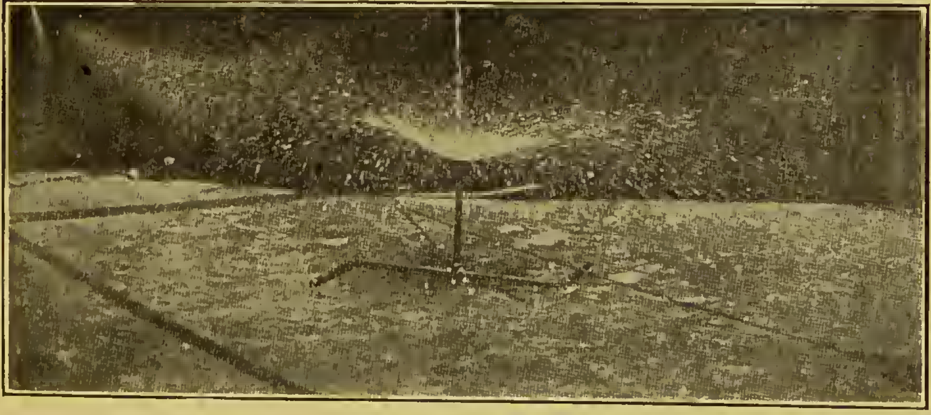


FIG. 100. Form of Spray from Technology Gravity Distributor.

Forms of Spray from Sewage Nozzles (Winslow, Phelps, Story and McRae, 1907).

cluding as independent variables the cone angle, orifice pressure, orifice elevation, blade length and nozzle pressure.

An entirely different type of fixed sprinkler devised at the Sewage Experiment Station of the Massachusetts Institute of Technology has distinct advantages for certain installations. Instead of spraying upward from closed pipes, the sewage in this system of gravity distribution drops from holes in the bottom of open troughs on to concave discs, or dash plates, from which it splashes upward and outward. The spray from one of these gravity distributors is shown in Fig. 100. Careful studies have indicated that a 3-inch disc with a curvature corresponding to a radius of 2 inches gives the best results. A total head of 4 feet between trough and filter is satisfactory; but the best distribution is attained with a total head of 6 feet, the disc itself being raised 2 feet above the surface of the bed (Winslow, Phelps, Story and McRae, 1907). The discs can best be placed about 11 feet apart, each taking a discharge of 4 gallons a minute for a rate of 2 million gallons per acre per day. This combination gives a distribution intermediate between that obtained with the small Birmingham nozzle and the large Columbus nozzle; but the smallest opening liable to clogging is larger than that of the Columbus sprinkler, and closed pipes beneath the surface of the bed are avoided. This device has been used with success at St. Anthony Park, Minnesota (Bass, 1908), and is to be used for a 2,800,000 gallon plant at Mt. Vernon, N. Y. (Robinson, 1909). The Mt. Vernon studies have shown that discs with the lips slightly turned over effect a much better distribution (Hammond, 1910).

Either the fixed sprinkler nozzle or the fixed gravity distributor will work well under fairly severe winter conditions, as shown by experience at the Columbus testing station, at the Technology experiment station and at the experimental plant at Worcester, Mass. Sprinkler nozzles at Worcester are shown in operation in Fig. 101, when the temperature was $-10°$ F. Deposits of ice and snow form between the heavily wetted circles which surround the individual sprinklers; but within each circle an ample area is kept

open by the warm sewage spray. In very severe climates, as in Canada, for example, the covering of trickling beds may prove necessary; and a covered plant for the Bordeaux prison near Montreal has recently been designed in this belief.

Unquestionably all forms of fixed sprinklers are inferior in point of evenness of distribution to good movable distributors of the English type. It is a matter of considerable importance

Fig. 101. View of Worcester Experimental Filter in Winter.

to determine how serious the inequalities may be with a sprinkler of a given type; and elaborate studies along this line have been carried out at the Technology experiment station in Boston (Winslow, Phelps, Story and McRae, 1907), at the Massachusetts State Board of Health Experiment Station at Lawrence (Gage, 1908), and at Waterbury, Conn., by Mr. W. Gavin Taylor (1909 *c*). The testing apparatus used at the Technology station consisted of a circular concrete drainage trough with a pipe for pressure nozzles at the center and a trough for operating

gravity distributors above. A galvanized iron box in the form of a sector of a circle revolved about the center and caught the discharge along different radii in a series of concentric 6-inch compartments. In the Lawrence experiments the spray was caught in a radial row of bottles, each bottle holding a 6-inch funnel, so spaced that the rims of the funnels were tangent to each other. The mechanical part of the tests is in any case very simple; but their interpretation is more difficult, because so many independent factors are involved. The absolute discharge in relation to the total wetted area must be considered, as well as the evenness of distribution within the wetted area, because with some nozzles the total discharge is far in excess of the amount which could be handled by the wetted area alone. The Technology and Lawrence workers have attempted to combine these various factors in arbitrary formulæ, neither of which can perhaps meet with general acceptance. It is probably best to confine the comparison of different sprinklers to the question of evenness of distribution within the wetted area, leaving the questions of head and rate and spacing of sprinklers to be separately considered. The evenness of distribution within the wetted area has been expressed by Phelps (1906) in a simple formula, as follows: The quantity, q, of liquid in each radial compartment (or in each bottle arranged along a radius) is multiplied by the distance, d, from the center, so as to give the total volume of liquid, v, discharged on the ring which the compartment represents. These quantities are then added to give the total volume, V, discharged on the whole circle. From this are derived the values, v', which would have been discharged on each ring if distribution had been perfect. From all the v' values in excess of the v values, the latter are subtracted, giving values of e, or excessive discharges. These added together give E, or the total excessive discharge, and Phelps takes for his coefficient unity minus the quotient of the excessive discharge divided by the total discharge.

$$C = 1 - \frac{E}{V}.$$

When the distribution is good, E is small and the coefficient approaches unity. A further correction must be made in practice, however, for the unwetted area between the circles. If the sprinklers are placed at the centers of squares, this means a waste of 22 per cent of the total filter area; but if they are arranged at the centers of hexagons, the waste area is reduced to 10 per cent.

The general results of the Technology tests are given in the table below. The gravity distributor was of the type described on page 342, operated under a 6-foot head. The pressure nozzles are the ones shown in Fig. 97. All were operated at a 6-foot head; but the Columbus nozzles gave slightly better results, under a 4-foot head (.65).

TABLE LXXXIII

COMPARATIVE EFFICIENCY OF FIXED DISTRIBUTORS

(Winslow, Phelps, Story and McRae, 1907.)

Type.	Rate (gallons per minute).	Sprinklers per acre, (2,000,000 gallons a day)	Coefficient.
Best gravity distributor	4.1	341	.76
Old Salford nozzle	2.9	483	.44
New Salford nozzle	2.1	667	.78
Birmingham nozzle	2.0	700	.84
Columbus nozzle	14.8	94	.61
Waterbury nozzle	10.4	134	.73

The Birmingham sprinkler nozzle with its small free opening (a $\frac{5}{64}$-inch ring) gives the most satisfactory distribution, but a large number of nozzles are required for a given area and the clogging of the small orifice calls for constant supervision. The gravity distributor gives a fairly even spray, and the Columbus nozzle, with its large orifice, naturally shows the poorest results. The serious unevenness of distribution with fixed sprinklers under constant heads is shown in Fig. 102, plotted by Taylor (1909 *c*).

Attempts have been made in two directions to better the results obtained from fixed sprinkler systems, by intermittent dosing, and by designing sprinklers to discharge a square rather than a circular spray. At Chesterfield in England the nozzles were

arranged to discharge intermittently, so as to spread the sewage more evenly over a wider area. At Columbus intermittent dosing was made a part of the design for two reasons. In the first place, the Columbus nozzle has so large an opening that the discharge under a constant head is far in excess of 2 million gallons per acre per day on the wetted area; intermittency corrects this difficulty, at the same time that it spreads the ring

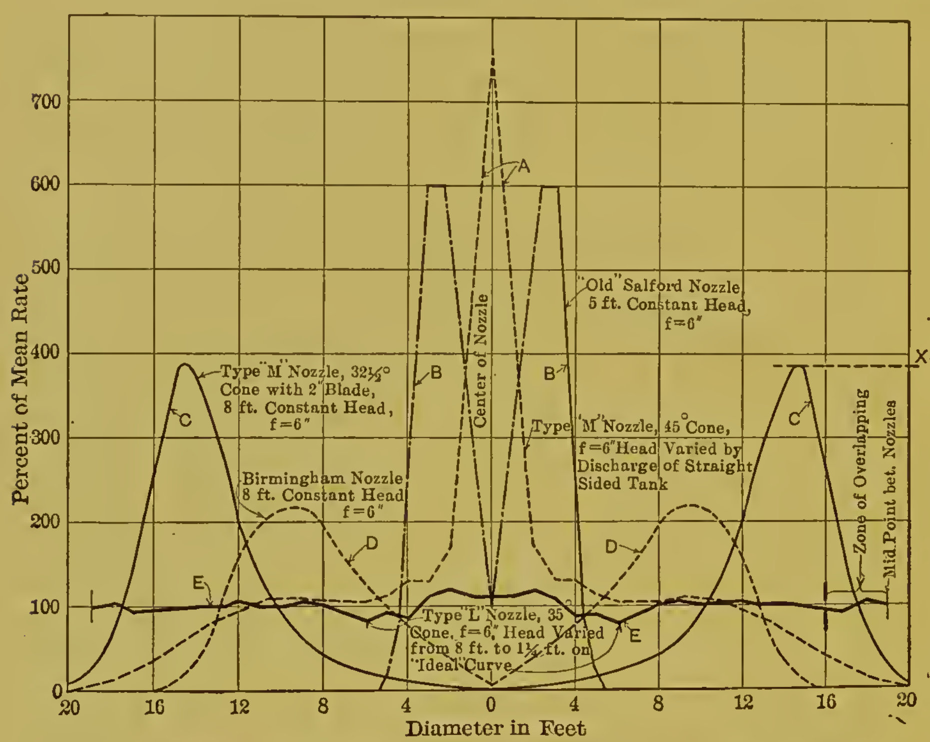


FIG. 102. Diagram Showing the Unevenness of Distribution from Fixed Sprinkler Nozzles (Taylor, 1909 *c*).

of discharge in and out, and thus improves the evenness of distribution. The latter end is not, however, always attained by simple intermittent operation. In the Technology experiments it appeared that when the Columbus nozzle was dosed from a siphon tank with a head varying between 4.3 feet and 2.5 feet the only effect was to lessen the size of the ring of excessive discharge. The coefficient with a constant head of 4.3 feet was .65, and with varying head it ranged from .45 to .56. On the

Columbus filters themselves it has finally been found best to discharge sewage on each bed in rotation under three different heads, — 4 feet, 7 feet and 9 feet (Fuller, 1909). The net result of this process is to secure excellent distribution, as shown in the table below. The coefficient calculated from the last column of figures would be .92.

TABLE LXXXIV

DISTRIBUTION OF SEWAGE OVER THE COLUMBUS FILTERS IN GALLONS PER SQUARE FOOT PER MINUTE

(Fuller, 1909.)

Distance from nozzles, feet.	Nozzle head.			Average for cycle.
	4 feet.	7 feet.	9 feet.	
1.0–1.5	.012	.011	.010	.011
1.5–2.0	.030	.012	.012	.018
2.0–2.5	.070	.017	.016	.034
2.5–3.0	.140	.025	.020	.061
3.0–3.5	.230	.037	.029	.098
3.5–4.0	.220	.053	.037	.103
4.0–4.5	.200	.074	.050	.108
4.5–5.0	.110	.098	.062	.090
5.0–5.5	.053	.124	.078	.085
5.5–6.0	.024	.136	.096	.085
6.0–6.5	.011	.137	.131	.091
6.5–7.0	.005	.108	.155	089
7.0–7.5	.002	.090	.175	089

W. Gavin Taylor, in his experiments at Waterbury, Conn., has devised an ingenious nozzle designed to cover a square field and thus do away with the waste area between sprinklers. The nozzle, shown in Fig. 103, has an inch opening, and through its center passes a standard which supports a conical spreader, with a vertical cut in each quadrant of the cone so that its upper surface has the shape of a clover leaf. The surfaces are so arranged as to discharge the sewage in a squarish sheet, and the nozzle is designed to be operated under varying head, so that the sheet spreads in and out from the center (Taylor, 1907 *a*). Mr. Taylor points out that a straight line variation in head could not be expected to produce even distribution. Theoretically, the head upon the nozzle should vary less rapidly near the maximum point than at

the minimum; and Mr. Taylor (1909 *c*) has designed a pressure undulating valve to secure it on a practical scale. In Fig. 102 he shows the results obtained with his nozzles (Types L and M) under the ideal pressure variation, under the variation in head obtained by discharge from a straight-sided siphon tank and under constant head, with similar curves for the Birmingham and Salford nozzles working under a constant head.

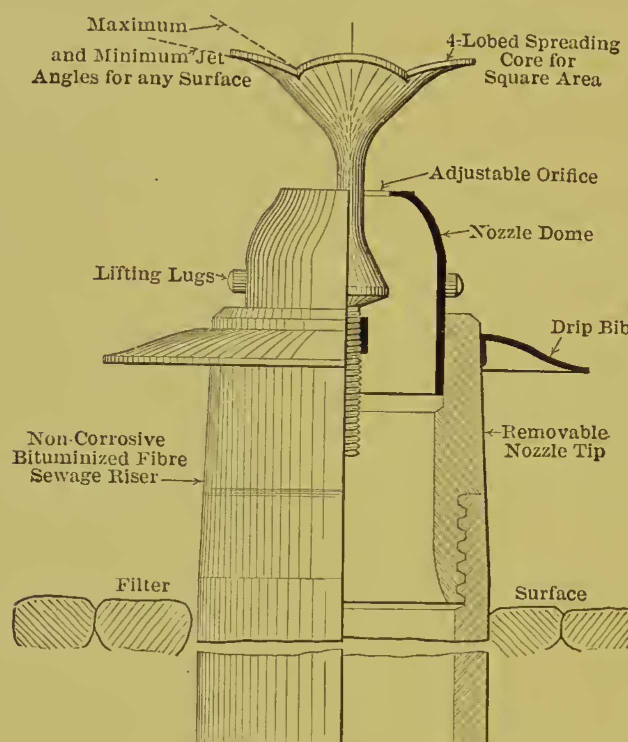


FIG. 103. The Taylor Nozzle for Covering a Square Area (Taylor, 1909 *c*).

At present the choice of a distribution system lies between the English moving distributors, which give an even discharge but are costly and cumbrous, and the fixed sprinkler nozzles and gravity distributors, which are comparatively cheap and simple in operation but give poorer distribution. It is not certain how serious the results of bad distribution may be in practice. At Birmingham Watson (1907) found that a bed dosed with a revolving Candy distributor gave a purification of

83.6 per cent against 80.4 per cent for a bed dosed with fixed sprinklers, — a rather insignificant advantage.

Results of Trickling Filtration. The general experience of the last ten years in England has shown very clearly that the trickling bed is an efficient and economical method of sewage treatment, and offers in general the most satisfactory process for communities where large areas of good sand are not easily available. The rates attainable are much higher than with any other method of treatment, being from two to four times as high as can be used with contact beds. In England, trickling rates generally vary from one to two million gallons per acre per day; in the United States the latter figure is generally taken as a safe minimum. In evidence before the Royal Commission, Ducat and Scott-Moncrieff recommend a rate of 1,200,000; Barwise suggests 1,500,000; Watson gives the figures quoted in the table below for current English practice. Still higher figures may be sustained for short periods. At Salford the rate, at first 3 million, was raised to 6 million without injuring the quality of the effluent.

TABLE LXXXV

DEPTH AND RATES OF TRICKLING FILTERS

(Watson, 1903.)

Place.	Depth.	Rate (million gallons per acre per day).
Leeds	9.0	1.2
Accrington	8.5	2.3
Birmingham	5.0	1.2
Hyde	9.0	2.6
York	6.5	2.6
Rochdale	9.0	2.3

The data collected by the Royal Commission may be expressed in tabular form as follows:

TABLE LXXXVI

RATE OF OPERATION. ENGLISH TRICKLING FILTERS

(R. S. C., 1908.)

Rate in million gallons per acre per day.	Number of filters in each class.	Rate in million gallons per acre per day.	Number of filters in each class.
.5-1.0	6	2.0-2.5	3
1.0-1.5	5	2.5-3.0	3
1.5-2.0	6	3.5-4.0	1

The analytical data for a number of English trickling filters are brought together in the table below. It will be noticed that the process here is a true nitrification, producing considerable amounts of nitrate in the effluent. The purification is good, distinctly higher in general than that obtained by the double-

TABLE LXXXVII

EFFICIENCY OF TRICKLING FILTERS

Parts per million. (Winslow and Phelps, 1906.)

Place.	Material.	Solids. Total.	Solids. Suspended.	Nitrogen as — Free ammonia.	Nitrogen as — Albuminoid ammonia.	Nitrogen as — Nitrates.	Nitrogen as — Nitrites and nitrates.	Oxygen consumed in 4 hours at 80° F.
Accrington *	Sewage	......	......	......	4.6	......	......	49.9
	Effluent	......	......	......	1.5	......	23.3	18.1
Hendon **	Sewage	......	......	71.6	13.2	......	......	147.0
	Effluent	......	......	2.5	.8	......	4.8	7.8
Hyde	Sewage	......	......	39.5	16.5	......	......	114.0
	Effluent	......	......	5.1	1.6	12	13.7	16.3
Leeds†	Sewage	1,120	187	21.2	5.1	......	......	57.5
	Effluent	1,000	80	8.1	‡1.3	7.8	......	‡9.8
Do §	Sewage	1,110	229	21.7	5.4	......	......	59.7
	Effluent	1,010	110	6.2	‡.9	9.6	......	‡8.4
Do.§§	Sewage	1,850	768	33.9	12.8	......	......	114.0
	Effluent	1,020	......	11.7	1.4	4.5	......	‡10.1
Do ‖	Sewage	1,820	850	32.8	12.6	......	......	141.0
	Effluent	986	......	1.9	.5	12.1	......	‡3.4
Do ‖‖	Sewage	1,470	486	23.5	9.4	......	......	116.0
	Effluent	979	81	3.2	1.3	6.2	......	12.1
Wolverhampton ¶	Sewage	......	......	47.1	3.3	1.4	......	43.3
	Effluent	......	......	23.8	.6	16.4	......	3.6
York¶¶	Sewage	840	......	31.8	5.9	......	......	42.0
	Effluent	719	......	2.1	.6	113.0	......	6.6

* Thermal aerobic filter. September 19 to October 19, 1898, receiving septic effluent (Rideal, 1901).

** Ducat filter, October 14, 1898, receiving crude sewage, single analysis (Rideal, 1901).

† Whittaker bed No. 1, March 9, 1899, to May 8, 1900, receiving septic effluent (Martin, 1905).

‡ Analysis made of the rough settling of suspended solids.

§ Whittaker bed No. 2, September 2, 1899, to January 30, 1900, receiving septic effluent (Martin, 1905).

§§ Ducat filter, March 29 to April 30, 1900, receiving crude sewage (Martin, 1905).

‖ Ducat filter, June 13 to July 7, 1900, receiving crude sewage (Martin, 1905).

‖‖ Leeds filter, December 13, 1900, to January 14, 1901, receiving crude sewage (Martin, 1905).

¶ Coal filter, January, 1896, to September, 1898, receiving chemical effluent (R. S. C., 1902).

¶¶ Septic effluent.

contact process, but not equal to that produced by intermittent filtration. In cold weather a distinct deterioration in the character of trickling effluents has been noted at Worcester, Mass.; and this should be expected from our knowledge of the effect of winter weather on intermittent filters.

TABLE LXXXVIII

COMPARATIVE STABILITY OF STORED EFFLUENTS FROM CONTACT AND TRICKLING FILTERS

(Clark, 1902.)

Time elapsed (days).	Nitrogen as — Free ammonia	Nitrogen as — Albuminoid ammonia. Total.	Nitrogen as — Albuminoid ammonia. In solution	Nitrogen as — Nitrates.	Nitrogen as — Nitrites.	Oxygen consumed in 2 minutes' boiling, corrected for nitrites.	Oxygen dissolved (per cent of saturation)
	Parts per million.						
Trickling Filter No. 135.							
0	20.1	2.3	1.0	51.8	.1	24.4	34.3
7	18.2	2.2	.8	44.2	6.0	19.5	15.3
14	19.4	1.9	.7	49.1	1.1	20.9	9.9
21	19.4	2.1	.8	46.2	1.3	21.5	13.2
28	20.1	2.0	.7	39.4	2.0	21.3	7.7
Trickling Filter No. 136.							
0	13.2	2.4	1.1	52.8	0.1	25.6	51.7
7	13.2	2.1	.7	50.8	1.1	19.4	15.5
14	13.2	1.9	.6	49.7	.5	18.3	5.0
21	13.2	1.8	.6	47.3	.2	17.1	1.6
28	13.5	1.9	.6	44.0	0.0	18.4	.3
Contact Filter No. 137.							
0	21.4	2.0	1.4	6.2	0.2	17.4	0.0
7	23.1	1.8	1.1	.1	0.0	22 8	0.0
14	23.9	1.6	.8	.1	0.0	24.8	0.0
21	26.4	1.4	.8	.1	0.0	23.2	0.0
28	26.4	1.3	9	1	0.0	23.2	0.0
Contact Filter No. 163.							
0	14.8	1.6	1.0	14.9	0.1	11.8	46 2
7	14.8	1.3	.6	6 6	0.0	10 8	0.0
14	15.1	1.1	.6	5.3	0.0	10.0	0.0
21	16.2	1.0	.6	1.6	0.0	9 4	0.0
28	17.4	9	.4	2.5	2	8 6	0.0

It must be remembered that the analytical results tell only a small part of the story in the case of the trickling filter. A trick-

ling effluent containing the same amount of free ammonia and organic nitrogen as a contact effluent will be far less putrescible, since the trickling filter effluent contains a large amount of oxygen, as dissolved oxygen and as nitrates, and this oxygen is available to meet the requirements of the reducing bodies left in the effluent. An experiment made by Clark (1902) illustrates this point. Two trickling effluents and two contact effluents were kept in the laboratory for a month in stoppered bottles, samples being withdrawn every week for analysis. The results are shown in the table on page 351. Initially the organic content of all four samples was about the same; but the trickling effluents contained large amounts of nitrates. One contact effluent contained dissolved oxygen at the start; but it lost it in a week and the nitrates fell to a low figure. In the trickling effluents, on the other hand, the dissolved oxygen was not quite exhausted even after a month, and a large fraction of the nitrates remained untouched. The contact effluents were putrefying and the trickling effluents were stable.

The success of the trickling filter must therefore be judged primarily by the stability of its effluent; and the results effected as a filter gradually ripens and becomes efficient may be illustrated by the results tabulated below from experiments at the Technology experiment station at Boston. The distribution upon this particular bed was quite imperfect until the fall of 1906.

TABLE LXXXIX

STABILITY OF TRICKLING FILTER EFFLUENTS

Percentage of samples decolorized at each specified period

Days		0–2	2–4	4–6	6–8	8–10	10–12	12–14	14+	Total samples.
Year.	Quarter.									
1906	1	40.5	18.9	2.7	5.4	8.1	0.0	0.0	24.4	37
1906	2	42.6	26.0	20.6	2.7	2.7	2.7	0.0	2.7	73
1906	3	23.7	38.9	15.3	5.1	6.8	0.0	0.0	10.2	59
1906	4 *a*	2.4	9.5	14.3	4.8	9.5	7.1	2.4	50.0	42
1906	4 *b*	2.9	11.4	2.9	8.6	11.4	2.9	8.6	51.3	35
1907	1	2.7	4.1	6.7	2.7	2.7	4.1	8.1	68.9	74
1907	2	4.4	2.9	1.5	0.0	1.5	2.9	1.5	85.3	68

One important advantage of the trickling process is the comparative freedom of the beds from clogging, due to the fact that their open construction and the unhindered flow of sewage permit whatever solid matter temporarily accumulates within to scale off and pass out with the effluent. The yearly cycle observed in the Boston experimental filters has been discussed on page 320; and it has been pointed out that in this case the annual output of suspended solids was practically equal to the amount applied. The same thing has been found true in actual practice at the Birmingham plant. In case this scaling process does not naturally take place, it should be accelerated by short resting periods, in the course of which the surface films dry out and become detached.

With filters of fine material ($\frac{1}{4}$ to $\frac{5}{8}$ inch) clogging takes place at or near the surface, so that surface renewal may be required. The Royal Commission found "examples of this at Salford, Hanley, Chesterfield, Birmingham and Hendon, though at none of these places was the choking serious. At Market Drayton and Leeds, however, the upper portions of the filters had to be washed or renewed after about three years' work." With coarser grain filling, on the other hand, trickling beds may be practically permanent. To quote again from the final report of the Royal Commission, "When coarse material is used in a percolating filter, there is apparently little danger of the filter becoming clogged, unless the sewage contains much fibre or is liable to give rise to fungoid growths, or unless serious disintegration of the material takes place. We have had no experience of such a filter becoming clogged with suspended matter when dealing with domestic sewage. At Accrington, coarse percolating filters have been in use for the treatment of septic tank liquor for eight years, and have not clogged. At Horfield, a coarse percolating filter treating chemical precipitation effluent and fed by means of dripping trays ran uninterruptedly until it was dismantled after five years' work. At York, coarse filters have been in use for the treatment of septic tank liquor, without interruption, for four years. There are, in fact, a number of such instances. This kind of filter,

Fig. 104. View of Clogged and Pooling Trickling Filter.

however, seems to be liable to choke as the result of surface growths. Where there is much fiber in the liquid to be treated (and the same thing would no doubt apply to grease and various trade wastes), percolating filters of even very coarse material may choke up. At Leeds, a percolating bed, which was constructed of

coarse coke, showed signs of becoming choked after having received septic tank liquor for eight and a half months. The primary cause of choking was, we think, the fibrous character of the suspended matter, but there was a considerable development of growth on the surface of this bed at the same time."

The causes which lead to fungous growths on the surface of trickling beds are not at present understood; but according to experience at Dorking by Houston and Colin Frye they can be easily got rid of by the application to the surface of the filter of a 20 per cent solution of caustic soda (one gallon to 6.4 square yards) (R. S. C., 1908). A curious minor nuisance observed at some of the English works (Accrington and Dorking) and at Reading, Pa., has been the cultivation in coarse trickling beds of vast multitudes of midges and flies. On account of this fact, and on account of the odors evolved from the sewage spray, trickling filters should, as a rule, be at least an eighth to a quarter of a mile from the nearest house or road.

Experience with Trickling Filters in England and Germany. Birmingham, the fourth city in England, with a population of about 950,000 under its main drainage board, is the one place in the world where the trickling filter has been most thoroughly studied. It holds the same relation to this process which Manchester occupies in regard to contact treatment. The sewage of Birmingham, about 1852, was conveyed by a main sewer into the River Tame, where it was discharged untreated. In 1859 experiments were begun at Saltley which led, in 1872, to the installation of chemical precipitation tanks and a sewage farm. In 1905 the total area of land in the hands of the Drainage Board was 2830 acres, of which 1784 were used for irrigation, and 35 acres for sludge disposal.

On the appointment of John D. Watson, M. Inst. C. E., as engineer to the Birmingham, Tame and Rea District Drainage Board, he began an elaborate series of large-scale experiments which have continued to the present time, and which, unfortunately, have never been reported fully in print. The results of these experi-

ments have led to the entire abandonment of chemical precipitation and sewage farming at Birmingham, and to the treating of the sewage on a strictly biological basis. In 1901 a beginning was made by converting the precipitation basins into septic tanks, and in 1903 the first trickling beds were built. At present the flow of about 30 million gallons is treated in a plant including the following elements: (*a*) sedimentation or detritus tanks; (*b*) septic tanks; (*c*) secondary sedimentation or silt tanks; (*d*) trickling beds; (*e*) separating tanks for removing suspended solids from the trickling effluent.

The Birmingham plant has recently been fully described by Watson (1910). The detritus tanks are 5 in number and have a total capacity of 500,000 gallons. The first septic tanks have a capacity of 6,730,000 U. S. gallons, a mean rate of flow of 1.2 feet per minute and a sedimentation period of 4.35 hours. The second septic tanks are 20 in number with a total capacity of 8,700,000 U. S. gallons, the storage period being 8 hours. These preliminary works are at Saltley, and here also are 30 acres of bacteria beds to be used chiefly as storm water beds. These are 6-foot trickling filters filled with clinker. From Saltley the septic effluent flows for 5 miles through a closed conduit to Minworth in the township of Sutton Coldfield where the trickling beds have been built. Here the septic effluent is first treated in the silt tanks mentioned above to reduce still further its suspended solids. These tanks (see Chap. IV) are 22 in number; the first six built are circular, 44 feet in diameter and 33.5 feet from the coping level to the bottom of the sump, the lower portion being in the form of an inverted cone having a batter of 1 to 1; the sixteen newer ones are 25 feet square and 20 feet deep to the apex of the pyramid. The septic effluent enters these newer tanks by a pipe dipping down to the middle of the tank, with a downward velocity of 1–2 feet per second. As it emerges from the mouth of the pipe it spreads out and ascends to the square portion of the tank at a decreasing velocity, which averages 7 feet per hour. The storage period in these tanks is about 4 hours. The sludge removed from

the bottom amounts to 1918 pounds of dry matter per million United States gallons of sewage treated and the cost of removing and burying it is about $14\frac{1}{3}$ cents per million gallons of sewage treated (Watson, 1907). The tanks are capable of effecting a reduction in suspended solids from 291 parts per million (the septic effluent value) to 61 parts per million; and as a matter of fact they did reduce the suspended matter in 1907 from 242 to 97; and the total cost is about 20 cents per million gallons.

The total amount of dry solid matter removed from the sewage is 67 long tons per day, or 1170 tons of liquid sludge containing 94.5 per cent water.

At first the sludge was spread in great shallow lagoons and allowed to dry sufficiently to be dug into the ground. This method was changed by discharging the sludge into trenches 3 feet wide and 18 inches deep, and covering it over with earth as soon as practicable. In 1907 the sludge was pumped to a field surrounded by earthen embankments 10–20 feet high, and although about 2 acres of this area were covered with sludge to a depth of 8 feet, and rapid decomposition of the organic matter was going on, as shown by the amount of gas continually evolved, no strong odor was noticeable from the adjoining banks.

The trickling beds at Sutton Coldfield occupy 30 acres. The beds are 1, 2 and 3 acres in area, with the exception of some quarter-acre and half-acre beds built for experimental purposes. The larger filters are rectangular in shape, with side walls of rubble, laid dry, and floors of concrete, with a fall of about 9 inches across the bed. Over the concrete is an aerating floor of semicircular stoneware tile, laid with loose joints. The filling material is 6–7 feet deep and consists of broken brick, slag, granite or quartzite $\frac{3}{4}$ to 2 inches in diameter. Mr. Watson's experiments led him to the conclusion that the nature of the material used was immaterial, so long as it was permanent and had a rough surface. Fine material, of course, gave somewhat better results; but the effluent from beds filled with 2-inch material was nonputrescible and the

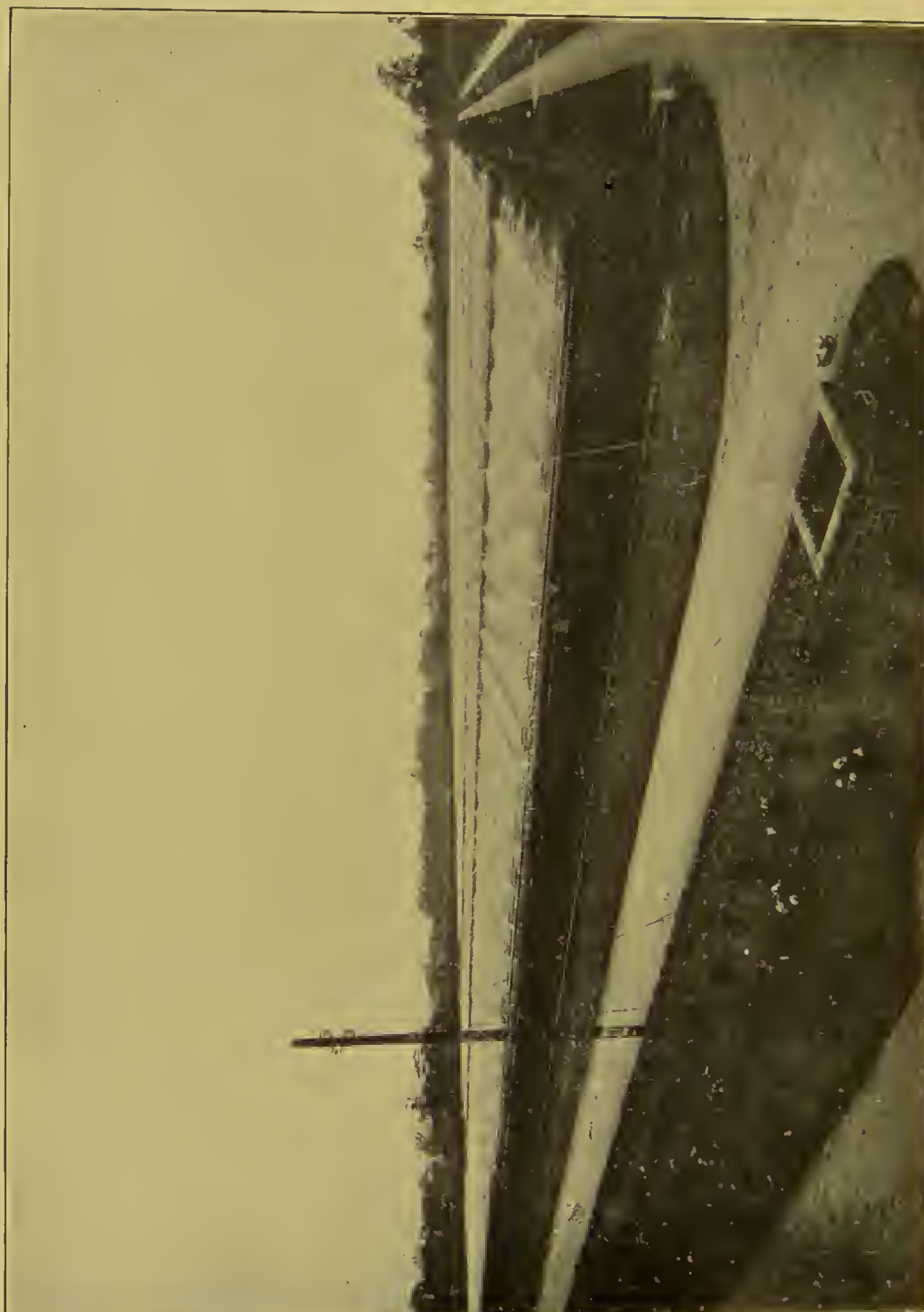


FIG. 105. General View of Birmingham Filters (courtesy of J. D. Watson).

cost of operation materially less. Thus the operation of a bed mainly of $\frac{1}{4}$–$\frac{1}{2}$ inch material with 1 foot of $\frac{1}{8}$–$\frac{1}{4}$ inch material on top and 2.25 feet of $1\frac{1}{2}$–$2\frac{1}{2}$ inch material below cost $13.60 per million United States gallons against $7.95 for a bed of $1\frac{1}{4}$–$2\frac{1}{2}$ inch material. The average rate of operation is 900,000 United States gallons per acre per day. Distribution is effected by means of the fixed sprinkler described on page 339. A general view of one

FIG. 106. View of Birmingham Beds in Winter (courtesy of J. D. Watson).

of the beds is shown in Fig. 105, and another view in winter in Fig. 106.

After filtration the trickling effluent contains about 125 parts per million of solids. This is treated in separator tanks at a cost of 28 cents per million United States gallons to eliminate solids; pumping sludge from these tanks and digging it into the land costs ten cents more (Watson, 1907).

Analytical results for the various parts of the Birmingham plant, in 1909, are given in the table on page 360.

TABLE XC

COMPOSITION OF SEWAGE AND EFFLUENTS AT BIRMINGHAM

Analyses of average weekly samples collected every half-hour during the 24 hours, in the year 1909.

Parts per million.

	Crude sewage.	Septic effluent.	Silt tank effluent.	Trickling effluent.	Separator tank effluent.
Oxygen consumed in 4 hours, unfiltered	277.4	200.4	113.2	39.9	21.3
Alb. ammonia	12.5	9.0	5.9	5.5	1.8
Nitrogen as nitrates and nitrites	......	......	....	11.3	15.7
Suspended solids	408.0	194.0	98.0	128.0	15.0

The average capital cost of the last eighteen beds built at Birmingham has been about $35,000 and the total cost of treatment taking the larger beds with their appurtenant tanks, etc., as a basis, and including all interest charges and operating expenses averages about $6.80, per million U. S. gallons.

Important investigations of the trickling process have been made at many other English cities besides Birmingham, the most notable of which, perhaps, are the studies at York (York, 1901). At several other large cities and at many smaller places, trickling filters are in operation, and the process is rapidly gaining in favor. Baker (1904) gives good descriptions of plants of this type at Salford, Accrington, and York, and Clark (1908) describes a visit to the filters at Blackburn, Haywood, Chesterfield, and Hanley. At Blackburn the use of soft material had led to disintegration and clogging, but at the other three plants excellent results were reported.

In Germany, too, the trickling filter is more and more clearly recognized as the most generally promising of biological methods. One of the most interesting trickling filter installations in this country may be seen at Wilmersdorf, a suburb of Berlin. The plant is designed for a maximum flow of 38,000,000 gallons per day, and includes septic tanks, trickling beds, secondary sedimentation basins for the trickling effluent and accessory sand filters. The septic tanks are six in number and have a total capacity of 3,178,000 gallons. The septic effluent passes to a gathering

chamber, from which it is discharged automatically on the beds. The latter are 56 in number, circular filters, each 65.6 feet in diameter. They are built on concrete floors but without walls, and are dosed by revolving sprinklers of the Barker Mill type. Figure 108, showing these beds, is from a detailed description of the plant by Gerhard (1908).

FIG. 107. General View of Salford Beds during Construction.

Trickling Filters in the United States. The general recognition of the trickling filter as a practical method of purifying sewage in the United States really dates from the experiments carried out by the city of Columbus, Ohio, in the years 1904 and 1905. A few small plants were in operation before that date (at Madison and West Allis, Wis., for example), but it was still uncertain to what extent the trickling process would operate successfully in the severe winter climate of the northern United States.

Columbus is a city of 125,000 inhabitants discharging its sewage into the Scioto River, a small stream quite incapable of

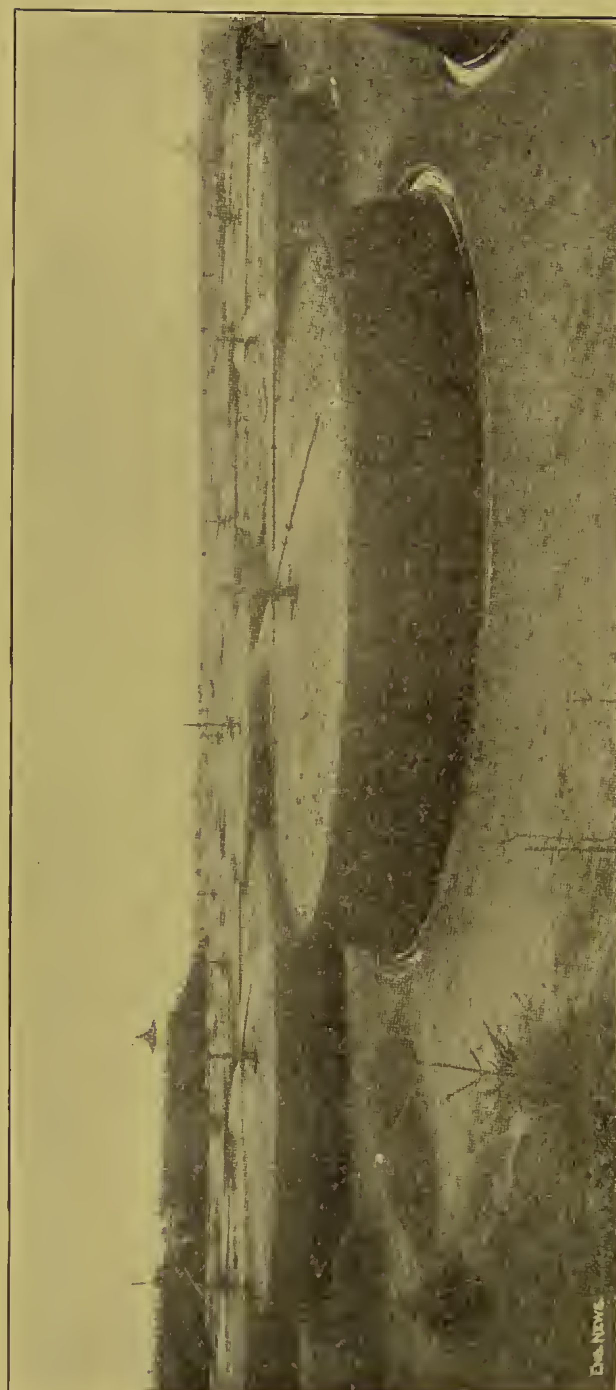


FIG. 108. Trickling Filters at Wilmersdorf, Germany (Gerhard, 1908).

handling it satisfactorily. Double filtration through coke had been suggested in 1898, septic treatment in 1900, and intermittent sand filters in 1901. In 1903, Messrs. Hering and Fuller of New York were called in consultation, and an experimental study of the problem was begun in the spring of 1904. The experiment station was under the immediate charge of Mr. G. A. Johnson. It included 7 tanks for preliminary treatment, 2 coke strainers, 6 contact beds, 6 trickling beds and 21 intermittent sand filters. These were studied with care during a period of one year's operation, and the results were discussed in an exhaustive report (Johnson, 1905). The final conclusion was that the trickling process could be operated with perfect success under the local conditions and that it would furnish the best solution of the problem.

At about the same time that this work was completed an outdoor trickling filter, 200 square feet in area, was put in operation at the Boston experiment station of the Massachusetts Institute of Technology, and two years later it was concluded that this process would offer the best means of treating sewage similar in character to Boston sewage (Winslow and Phelps, 1907). At Worcester extensive experiments with trickling filters have also been made on a comparatively large scale, and the results compared with those obtained with the same sewage on contact beds. The results show that for Worcester sewage trickling filters could be used to much greater advantage than contact beds. In 1905, an experiment station was designed for the city of Waterbury, Conn., under the direction of Mr. R. A. Cairns, and a valuable series of experiments has been carried out by Mr. W. Gavin Taylor (Taylor, 1907 *b*). A similar experimental plant was built at Baltimore, Md., in 1907 (Eng. News, 1907), under the direction of Mr. Calvin W. Hendricks and in immediate charge of Mr. E. B. Whitman. Neither of the last two investigations has been published in detail, but both have led to the adoption of the trickling process.

The first large trickling plant actually constructed was built at Reading, Pa., by Messrs. Hering and Fuller, in coöpera-

tion with the city engineer, Mr. Elmer H. Beard, and with Mr. O. M. Weand, and was put in operation in February, 1908. The present plant has a capacity of 2 million gallons a day, and includes careful preliminary screening, septic treatment, trickling filters and final sedimentation. Screening is made particularly thorough on account of the fibrous material contributed by certain factory wastes. The device in use has already been described in Chapter III. After screening, the sewage is pumped to the septic tank on the opposite side of the Schuylkill River, a distance of 6600 feet. The septic tank is open and of concrete 51 feet 8 inches wide, 253 feet long and 16 feet deep to the flow-line, the unusual depth being due to the fact that the steel framework of an earlier disposal plant was used in its construction. The capacity of the tank is 1,600,000 gallons, or 19 hours' flow. The influent enters the tank from 12 5-inch holes in a 14-inch steel pipe running across one end; the effluent passes over a weir at the other end. Five inches in front of the weir a scum-board extends 1 foot below the crest of the weir.

The trickling filters, two one-acre beds, are on Fritz's Island in the Schuylkill River, 2500 feet from the septic tank. The filling material is blast furnace slag, broken by hand to a size between $1\frac{1}{2}$ and 4 inches. The outside of the filters is formed of vertical walls of dry rubble for a short distance up, and above, of large pieces of slag laid by hand to a batter of about 2 horizontal to 3 vertical. The depth of the filters is 5 feet. The sprinkling nozzles are of the general Columbus type, but modified by supporting the spreading cone on a rod passing up through the orifice (as in the Birmingham sprinkler), thus avoiding the interference due to the supporting side arms of the Columbus nozzle. The cone has 45 degree sides; the supporting rod has a diameter of $\frac{5}{16}$ inch and the orifice itself is $1\frac{5}{6}$ inch in diameter. The sprinkler heads are set at intervals of 14 feet 2 inches on parallel lines of 8-inch pipes, laid 13.7 feet apart a short distance above the floor of the filter. The distribution system is dosed from a siphon tank, so arranged that the nozzles work under a head varying from 6 feet to 1.4 feet, a complete cycle occupying about 11 minutes. The dis-

tribution which results is still somewhat uneven, as indicated by the table below:

TABLE XCI

DISTRIBUTION EFFECTED BY READING SPRINKLERS

(Fuller, 1909.)

Distance from nozzle in feet.	Gallons per square foot per minute.	Distance from nozzle in feet.	Gallons per square foot per minute.
1	.007	4	.074
2	.035	5	.066
3	.066	6	.043

The effluent from the trickling beds is settled in a basin 100 feet by 95 feet 9 inches in area and varying in depth from 4 to 5 feet. The capacity is 340,000 gallons, or one-ninth of the daily flow. The analytical results for the 14 months, February, 1908, to March, 1909, are given below:

TABLE XCII

ANALYSES OF SEWAGE AND EFFLUENT AT READING

Parts per million. (Fuller, 1909.)

	Suspended solids.	Oxygen consumed.	Nitrogen as nitrates.	Dissolved oxygen.	Bacteria per c.c.
Screened sewage	165	57	....	....	3,100,000
Septic effluent	43	26	....	....	1,800,000
Filter effluent	....	....	4.5	....	600,000
Settled effluent	20	15	5.0	6.5	670,000

The second large trickling filter in the United States was completed in the fall of 1908 at Washington, Pa. The dry-weather flow is about one million gallons per day, and the sewage from 1904 to 1908 was very imperfectly treated in a septic tank and crude stone strainer. The new plant, capacity two million gallons, was designed by one of the authors and was built under his supervision by Mr. D. M. Belcher as Resident Engineer. It includes as its main elements a screen chamber with two inclined screens having $\frac{5}{8}$ and $\frac{1}{4}$ inch mesh, described in

Chapter III; four open concrete septic tanks with a total capacity of 800,000 gallons; trickling filters; and a final sedimentation basin. The four filters are each 100 by 150 feet in superficial area (see general plan in Fig. 109). Walls and floor are of concrete

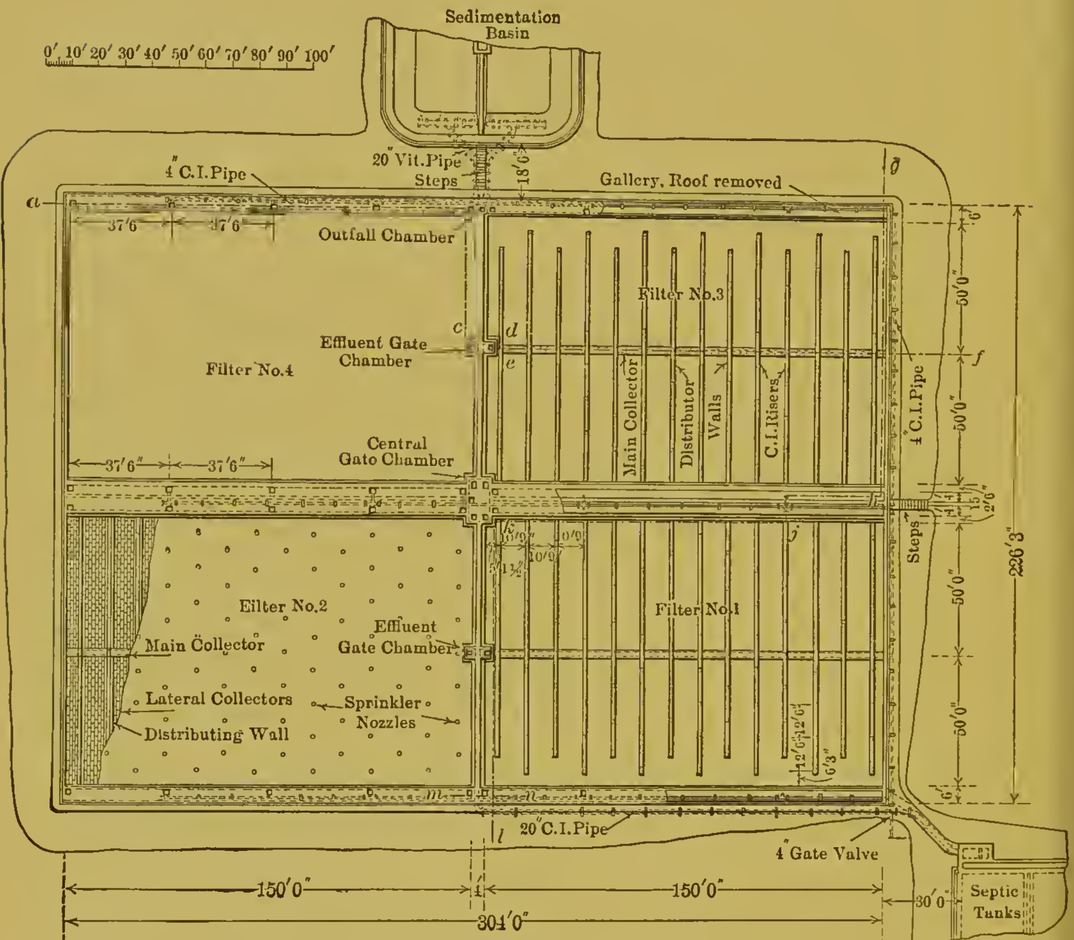


FIG. 109. Plan of Trickling Beds at Washington, Pa.

and the filtering layer consists of 6 feet 10 inches of broken stone between 1 and 4 inches in diameter. The bulk of the bed was made of a low-grade local limestone with 6 inches of harder stone on the top. Sewage is sprayed over the surface by nozzles of the Columbus type, placed 12.5 feet apart on lines 10.75 feet apart. They are operated with a head varying between 7 feet and 2 feet,

the average discharge being 10 gallons per minute. The filters are underdrained with half-tile, 61,000 feet of pipe being used for the 1.5 acre of filter.

The plant at Columbus, Ohio, which first went into operation in November, 1908, is the largest and in some respects the most interesting of American trickling filters. The capacity of this plant is 20,000,000 gallons, and it was designed under the direction of Messrs. Hering and Fuller of New York, consulting engineers, and Mr. Julian Griggs of Columbus, chief engineer. Mr. John H. Gregory was engineer of design and construction, and he has recently published an excellent description of the plant (Gregory, 1910).

The sewage is roughly screened at the pumping station by bar screens, and thence passes to the purification works, where it is treated in primary and secondary septic tanks constructed of reinforced concrete. The primary tanks are four in number, with a total capacity of 2,840,000 gallons; the secondary tanks are two in number, and each one has a capacity of 2,590,000 gallons (Fig. 45). Septic action, as observed in June, 1909, was very violent in the primary tanks and slight in the secondary tanks. Scum formation was slight, but sludge accumulates in the primary tanks at a rate of about 4 feet in 3 months. In the controller well is a device for maintaining a constant flow to the filters, variations in pumping being taken up by the septic tank. The controllers may be adjusted by a system of weights to deliver any quantity between 10 and 22 million gallons per day. From the controller well the sewage passes by 42 inch gates to the distributing well, and thence by 24 inch by 36 inch gates into the manhole chambers and filter distributors. The effluent passes from the main collectors into the sump wells, and thence by 24 inch by 36 inch gates into the effluent well and the effluent conduit. There are in all forty-two sluice gates, all operated by hand.

The six filters, four of which have been constructed for present use, are designed as equilateral triangles about 500 feet on a side, radiating out from the central gatehouse. Each filter has an area of 2.5 acres, and all are constructed of concrete and steel.

Sewage is distributed to each half-filter by a 30-inch main of reinforced concrete supported by cross walls which carry 5 and 6 inch lateral distributors. These laterals are 13 feet $9\frac{7}{8}$ inches apart and carry risers spaced 15 feet 4 inches between centers. Each set of three sprinkler nozzles forms an equilateral triangle 15 feet 4 inches on a side. The general arrangement of main and lateral distributors and underdrains is shown in Fig. 110.

FIG. 110. View of Columbus Filters during Construction (courtesy of J. H. Gregory).

The sprinkler nozzles are of the general Columbus type, shown in Fig. 97, with $\frac{9}{16}$ nozzle openings. Each orifice is rated to discharge 13.5 gallons per minute under a 5-foot head, and there are 211 nozzles to the acre. In order to secure more even distribution each bed, or half-bed, was dosed in 1909 for successive periods under three different heads 4 feet, 7 feet and 9 feet; and between each period the bed was given a period of entire rest. The net result of this mode of operation is that the beds are

dosed at a rate of about 4 million gallons per acre for half the time, resting for the other moiety.

The filtering layer itself is 5.5 feet deep, the lower 10 inches being 3–4 inch limestone and the rest 1.25–3 inch material of the same sort. 80,125 cubic yards of filling were used, costing in place $1.57 per yard. The floor of the filter is of 4-inch concrete and is practically covered with 6-inch notched half-tile, bedded about $\frac{1}{4}$ inch into the concrete, while it was soft (see Fig. 110). There are about 100 miles of this tile in the entire plant. A general view of one of the filters is shown in Fig. 111.

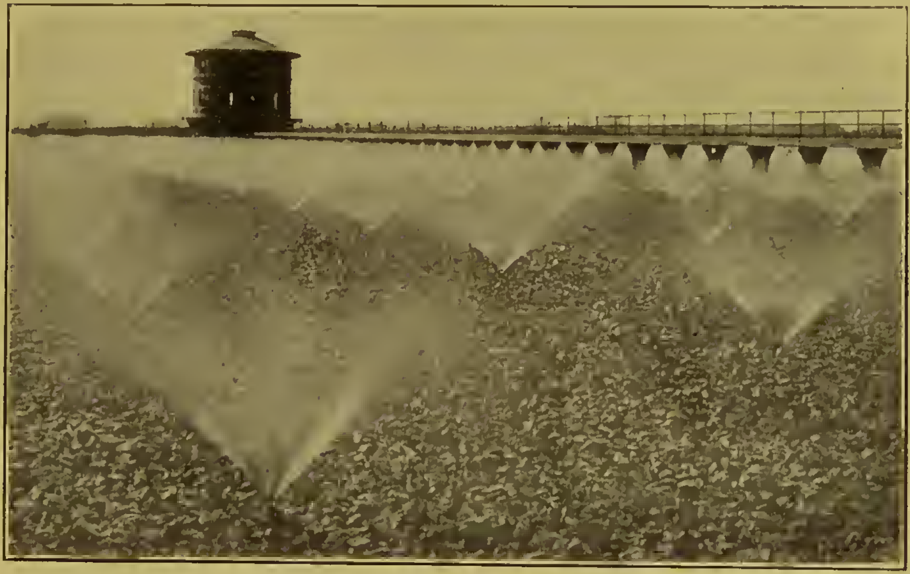


Fig. 111. View of Columbus Filters in Operation (courtesy of J. H. Gregory).

The effluent from the trickling beds is finally settled in two settling basins, each having a capacity of 2,000,000 gallons. The sludge from these basins is pumped out and discharged into the Scioto River at times of high water; and when the river is above a certain height the trickling effluent itself is discharged directly into the river without preliminary sedimentation.

The total cost of the Columbus purification plant, exclusive of land, was $440,000. It is operated by a chemist in charge, an assistant chemist, two laborers and a night man. It requires about a quarter of the time of one man to keep the sprinkler nozzles in good working order. The crude sewage and septic effluent were sampled, in 1909, every 2 hours, the filter effluent every 15 minutes, and the composite samples analyzed once a day. Bacterial samples were examined twice and putrescibility tests made 4 times a day. The settling basin effluent by itself was frequently putrescible during the spring of 1909, but even at that time it was stable when diluted with an equal volume of river water.

The principal analytical data are indicated in the table below:

TABLE XCIII

ANALYSES OF SEWAGE AND EFFLUENTS AT COLUMBUS, OHIO, 1909

Parts per million.

	Screened sewage.	Septic sewage.	Effluent filter.	Settled effluent.
Suspended solids	200	82	84	41
Oxygen consumed	61	38	21	19
Bacteria, per c.c.	2,370,000	1,050,000	560,000	740,000

The Removal of Suspended Solids from Trickling Filter Effluents. Where a sewage effluent is discharged into a rather large volume of water, and particularly in a region of strong currents, the effluent may be satisfactory just as it comes from the trickling bed. Such is the case at Columbus when the Scioto River is high; and such will often be the case with maritime towns. Where the effluent must be discharged into a small stream or a shallow bay, supplementary sedimentation for the removal of suspended solids is essential. The effluent as a whole may be stable; yet if the solids settle and accumulate on the bottom of a stream,—by themselves and in the absence of the ample supply of oxygen carried by the liquid, they may prove putrescible.

Secondary sedimentation forms, therefore, an integral part of most trickling filter plants.

Fortunately, the solids in the trickling effluent are in comparatively dense masses and settle out very readily. As a rule, with a two-hour storage period a removal of one-half to two-thirds of the suspended matter present is easily effected. The data for the Birmingham, Columbus, and Reading plants and for the experimental filters at Boston are brought together in the table below.

TABLE XCIV

SEDIMENTATION OF TRICKLING EFFLUENTS

	Flow period, hours.	Suspended solids. Parts per million.	
		Trickling effluent.	Settled effluent.
Birmingham	....	65	18
Columbus	5.7	84	41
Reading	2.6	....	20
Boston	2.0	117	57

The weight of opinion, both in England and in this country, is in favor of securing a fairly complete removal of suspended solids before treatment on trickling beds. Thus there is generally involved a double system of sedimentation, preceding and following treatment on the beds. In certain cases, when a rather weak sewage is treated on beds of coarse material, it may be possible to dispense with the preliminary treatment entirely, applying crude sewage to the filters and removing suspended solids once after organic stability has been attained. The table on page 372, for example, shows the result of two years' experiments at Boston. The final effluent from the process, including septic treatment, was of course better than the other, having 50 parts of suspended solids against 65. On the other hand, the free ammonia was higher in this effluent and the organic nitrogen and oxygen consumed only slightly lower. Both filter effluents, even without sedimentation, were stable for four days 90 per cent of the time.

TABLE XCV

PURIFICATION OF BOSTON SEWAGE WITH AND WITHOUT SEPTIC TREATMENT, 1905–1907

Parts per million. (Winslow and Phelps, 1907.)

	Turbidity.	Sediment.	Suspended solids. Total.	Suspended solids. Fixed.	Nitrogen as – Organic nitrogen	Nitrogen as – Free ammonia.	Nitrogen as – Nitrates.	Oxygen consumed.	Oxygen dissolved.
Crude sewage	279	121	135	44	9.1	13.9	.2	56	3.4
Trickling effluent	200	136	138	58	7.1	10.4	4.4	41	7.9
Settled effluent	128	62	65	24	4.1	10.2	4.6	31	......
Septic sewage	213	87	81	18	6.5	17.5	0.0	57	0.0
Trickling effluent	147	78	96	37	5.8	12.6	4.7	34	7.6
Settled effluent	106	51	50	18	3.6	12.6	4.8	29	......

Comparative Costs of Trickling Filtration and Other Processes. In general it seems clear that treatment on trickling beds is likely to prove the most economical method of purifying sewage to the point of organic stability, where the presence of suspended matter is not an objection. In construction the great point in favor of the trickling bed is the smaller area required. The table below, from results obtained at Croydon, gives an excellent idea of the comparative efficiency of unit areas:

TABLE XCVI

COMPARATIVE EFFICIENCY OF TRICKLING AND CONTACT BEDS

(Farmer, 1909.)

	Contact beds, 4 years.	Trickling beds, 2 years.
Total gallons treated	892,228,000	962,496,000
Gallons per square yard per day	61	173
Gallons per cubic yard per day	46	104
Per cent reduction oxygen consumed	47.3	76.8
Per cent reduction albuminoid ammonia	40.6	73.8

One acre of intermittent sand filter will handle the sewage from 500–1000 persons; one acre of a double contact system will suffice for 4000–5000 people; while an acre of trickling surface can purify the sewage from a population of at least 10,000.

The cost of distributing apparatus is of course a serious consideration in the case of the trickling filter, but this is seldom sufficient to offset the advantage of the greatly diminished area.

In regard to operating cost, the advantage is generally in favor of the trickling process. Intermittent filters are expensive, if properly cared for, and although the ordinary maintenance of the contact bed may be slightly less than that of the trickling bed, this is outweighed by the necessity for washing and replacing filtering material which appears not to be required in the trickling process.

It is difficult to make general statements in regard to costs which are so markedly affected by local conditions as those involved in sewage purification work. Rather careful studies based upon certain detailed assumptions have, however, been made by high authorities, in England and America, which may safely be taken as representing the relative expense of contact and trickling treatment.

In the final report of the British Commission on Sewage Disposal (R. S. C., 1908), Mr. G. B. Kershaw presents comparative cost data for a plant to treat a dry-weather flow of one million gallons a day of average domestic sewage, which led the Royal Commission to the final conclusion that "on the basis which we have adopted, purification of sewage (after preliminary treatment) by means of percolating filters costs only about two-thirds as much as purification by double contact beds. Where, however, the sewage has first been subjected to quiescent settlement, with chemicals, and single contact is sufficient to produce a satisfactory effluent, the cost becomes more nearly equal, though percolating filters are, even in that case, slightly cheaper."

A somewhat similar idea of comparative costs under American conditions may be obtained from the estimates prepared for the International Waterways Commission by Messrs. Hering and Fuller of New York in regard to the disposal of the sewage from the Calumet district of Chicago. These figures were made on the basis of a daily flow of 156 million gallons and include estimates for intermittent filtration as well as contact and trickling treatment. The cost of intercepting sewers and pumping stations is included because this factor is directly affected by the method of treatment

chosen, intermittent filtration requiring a much longer sewer in order to reach suitable sand areas. In preparing the table below the figures for annual operation given by Messrs. Hering and Fuller have been reduced to a million gallon basis and interest on their construction charges has been calculated at 5 per cent. As a matter of fact, the costs should of course be proportionately higher for a small plant than for the one contemplated in the original estimates. The area allowed for 156 million gallons was 1200 acres in the case of the sand filters, 300 acres for contact beds and 80 acres for trickling beds.

TABLE XCVII

COMPARATIVE COST OF SEWAGE TREATMENT AT CHICAGO BY VARIOUS METHODS

Cost per million gallons. (Hering and Fuller, 1907.)

	Intermittent filters.	Contact beds.	Trickling beds.
Construction charges:			
Intercepting sewers, pumping stations and appurtenances	4.46	2.90	2.90
Septic tanks, covered, including sludge disposal facilities	.83	.83	.83
Filters, office, laboratory, etc.	3.16	5.27	3.16
Settling basins	....	....	.18
Contingencies and supervision, 15 per cent	1.27	1.35	1.06
Total	9.72	10.35	8.13
Operation costs.			
Pumping, fuel, labor and repairs	5.26	3.52	3.52
Supervision, analytical and clerical assistance	.44	.53	.53
Care of septic and settling tanks, including sludge disposal	.63	.63	.95
Care of filters	8.42	4.56	1.93
Supplies and miscellaneous	.44	.44	.44
Total	15.19	9.68	7.37
Total cost	24.91	20.03	15.50

Of course the English and American estimates are not directly comparable, since each includes items left out in the other and since the unit costs are widely different. The relative costs of the different processes are, however, strikingly concordant. In each case the cost with contact treatment is between 40 and 50 per cent higher than with trickling beds; and intermittent filtration appears from the Calumet estimates to be still more costly than contact treatment.

CHAPTER XII

DISINFECTION OF SEWAGE AND SEWAGE EFFLUENTS

The Removal of Bacteria from Sewage. The primary object of sewage purification is the oxidation of organic matter, — its conversion into a stable form, so that it will not putrefy and create a nuisance. This end can often be attained by methods which do not effect a very notable reduction in the number of bacteria present. In many cases such methods are well suited to local conditions. Where, for example, the effluent from a purification plant is discharged into a small stream which is not used as a source of water supply, organic stability and removal of suspended matter is all that can properly be demanded. In other instances, however, bacterial purification may be required. Where an effluent is discharged into the ocean in a strong current of water, suspended solids and organic matter may prove inoffensive; but if there be important shellfish beds in the neighborhood, bacterial purification is an imperative necessity.

The relative importance of freedom from solids, organic stability and bacterial purity must be determined in each particular case by a study of local factors. The principal conditions which indicate the need for an effluent, low in bacteria, may be briefly summarized as follows:

a. When a stream is used without purification as a source of water supply the removal of bacteria is obviously essential, and the same is true, though to a less degree, in the case of sewage discharged in any large amount into lakes of the same character.

b. Even when the water taken from a stream is purified before it is used, it may be desirable to remove bacteria at a sewage outfall above, if the water intake is in the near neighborhood of the sewage outfall and if the amount of pollution is considerable. The

character of a raw water determines the rate of filtration and the cost of the process. It is not fair to place an abnormally heavy burden upon one community as a result of the negligence of another.

c. It may at times be proper to purify sewage bacterially before it is discharged into a body of water used extensively for bathing.

d. It is probably best under all conditions to provide sterilizing treatment for specially infected sewage (like that from a contagious disease hospital) before it is discharged into any stream.

e. The most important case, in practical magnitude, is the case in which shellfish layings are threatened by the discharge of sewage into tidal waters. In this case there is no alternative except the abandonment of the shellfish industry or the bacterial purification of the sewage which menaces it. All along the Atlantic seaboard this is a problem of pressing moment.

The Bacterial Efficiency of Sewage Filtration. There is a wide difference in bacterial efficiency between the older processes of sewage disposal, irrigation and intermittent filtration, on the one hand, and the newer processes of the contact bed and the trickling filter, on the other. Figures have been quoted in Chapter VIII (page 206) which show that, while most of the English sewage farms yield effluents containing bacteria in the hundred-thousands, the Nottingham effluent shows frequently less than 1000 bacteria per c.c. In Chapter IX it has been shown that good intermittent sand filters yield still better results. At Ames, Iowa (page 262), and at Brockton, Mass. (page 253), the bacterial reduction is well over 99 per cent and the actual number of bacteria in the effluents is generally under 10,000 per c.c. Such results as these indicate that the intermittent sand filter, when properly constructed and properly operated, will secure a reasonable bacterial purification as well as an organically stable effluent.

With the newer types of coarse-grain filters conditions are very different. All these processes produce a certain reduction in bacterial numbers; but the decrease is not sufficient to be of very great sanitary moment. With a material containing so many

bacteria as sewage, a reduction of 99 per cent may be satisfactory, but a reduction of 90 per cent, leaving several hundred thousand bacteria in the effluent, is certainly not satisfactory. That no better result than this can be expected from the contact bed or trickling filter may be made clear by a few figures quoted from the cases collected by Prescott and Winslow (1908):

"In the Columbus experiments, Johnson (1905) found from one to two million bacteria in the effluents of contact beds and from 750,000 to 1,900,000 in the effluents from trickling filters. The average percentage reduction effected by seven contact beds and six trickling filters is shown below:

TABLE XCVIII

REDUCTION OF BACTERIA AT COLUMBUS, OHIO

(Johnson, 1905.)

Contact beds.	Per cent reduction.	Trickling filters.	Per cent reduction.
Primary A	60	A	74
Primary B	43	B	70
Primary C	33	C	70
Primary D	33	D	69
Primary E	0	E	46
Secondary A	38	F	21
Secondary B	39	....	....

"Thumm and Pritzkow (1903), at the Berlin Experiment Station, obtained the results tabulated below:

TABLE XCIX

BACTERIA IN SEWAGE, CONTACT EFFLUENT AND SAND EFFLUENT AT BERLIN

	Bacteria per c.c.
Crude sewage	16,900,000
Primary contact effluent (coarse coke)	12,400,000
Secondary contact effluent (fine coke)	5,600,000
Tertiary sand effluent	1,100,000
Primary contact effluent (fine coke)	7,400,000
Secondary sand effluent	1,800,000

"At the experiment station of La Madeleine, in Lille, Calmette (1907) reports 5,000,000 bacteria per c.c. in the crude sewage, 2,900,000 in the second-contact effluent and 800,000 in the effluent from the trickling bed. Of 20,000 *B. coli* per c.c. applied to the filters, the contact system delivered 4000 and the trickling bed 2000 per c.c.

"The average results of examinations made three times a week at the Sewage Experiment Station of the Massachusetts Institute of Technology, during two different periods, were as follows:

TABLE C

BACTERIA IN SEWAGE, SEPTIC EFFLUENT AND TRICKLING EFFLUENTS AT BOSTON

(Winslow and Phelps, 1907.)

	Bacteria per c.c.				*B. coli* Positive tests in .000001 c.c.*
	July-Sept., 1906.		Oct., 1906–April, 1907.		July–Sept., 1906.
	No.	Per cent reduction.	No.	Per cent reduction.	Per cent positive.
Sewage	1,300,000	....	1,200,000	....	65
Septic effluent	1,650,000	Inc.	750,000	38	66
Effluent from trickling bed	750,000	42	200,000	83	35
Effluent from septic tank and trickling bed	750,000	42	180,000	85	35

* Jackson bile test.

"The contact beds, as operated on a practical scale in England, show considerably higher numbers. At London the Barking and Crossness beds yielded effluents containing one to five million bacteria per c.c., of which 100,000 to 600,000 were *B. coli*.

"There are few plants of the newer types now in operation in the United States, and fewer still are controlled by bacteriological examinations.* At Plainfield, N. J., however, the combination of septic tank and double-contact beds produces a bacterial purification of 80 to 90 per cent as measured by total numbers. The following table shows the results of four examinations made in 1906:

* Figures for Reading and Columbus are cited on pages 365 and 370.

TABLE CI

BACTERIA IN SEWAGE, SEPTIC EFFLUENT AND CONTACT EFFLUENT AT PLAINFIELD, N. J.

(N. J., 1907.)

Date.	Sewage.	Bacteria per c.c.		*B. coli* in —		
		Septic effluent.	Secondary contact effluent.	Sewage.	Septic effluent.	Secondary contact effluent.
				c.c.	c.c.	c.c.
July 9.......	2,295,200	659,200	591,300	.00001	.00001	.0001
July 9.......	2,043,300	555,000	172,600	.000001	.0001	.0001
August 9....	1,371,700	989,700	186,700	.00001	.0001	.0001
August 9....	1,655,000	....	338,000	.000001	....	.00001

"It is obvious that effluents of this character cannot be considered satisfactory from the standpoint of bacterial purification. As Houston concluded, after a careful review of the subject, 'The different kinds of bacteria and their relative abundance appear to be very much the same in the effluents as in the crude sewage. Thus, as regards undesirable bacteria, the effluents frequently contain nearly as many *B. coli*, proteus-like germs, spores of *B. enteritidis sporogenes* and streptococci, as crude sewage. In no case, seemingly, has the reduction of these objectionable bacteria been so marked as to be very material from the point of view of the epidemiologist' (Houston, 1902).

"Experimental studies with specific bacteria have confirmed these conclusions. Houston (1904 *b*) found that *B. pyocyaneus* appeared in the effluent of a trickling bed ten minutes after application to the top and continued to be discharged for ten days. In septic tanks and contact beds, the same germ persisted for ten days. Rideal (1906) quotes experiments by Pickard at Exeter, which show that typhoid bacilli may persist for two weeks in a septic tank and that contact bed treatment only effects a 90 per cent removal of these organisms."

Clearly, where bacterial purity is required, the contact bed and the trickling filter must be supplemented by some special process of bacterial removal.

Possible Methods of Disinfecting Sewage. A wide variety of processes have been suggested for disinfecting sewage and sewage effluents, particularly in England. Most of them are discussed with some fullness by Rideal (1905), and with calculations as to

American costs by Phelps and Carpenter (1906). They may be grouped for convenience under five heads: (1) heat; (2) caustic lime; (3) acids; (4) metallic salts; (5) oxidizing agents.

1. *Heat.* The use of heat has been suggested by Klein (R. S. C., 1902). The sewage was to be treated by a patented apparatus with the recovery of ammonia. Phelps and Carpenter calculated that to raise sewage to the boiling point would require 40 tons of coal per million gallons, at a cost of $160. At present this process is certainly not practical, although recent advances in the sterilization of drinking water by heat have shown that surprising economies may be effected by properly designed apparatus.

2. *Caustic Lime.* Caustic lime has an important application in sewage purification, for the removal of suspended solids, as pointed out in Chapter V. It is one of the best precipitants in use, and in the course of precipitation a large number of bacteria are mechanically removed. Lime has also a certain direct disinfectant action. This action is only slight, however. Rideal found that 1000 parts per million failed to produce satisfactory bacterial purification.

3. *Acids.* Acid substances, as a class, exert a much stronger disinfectant action than alkalis. Some have a specific poisonous effect of their own, like carbolic and benzoic acids; and the mineral acids uniformly exercise disinfectant properties by means of their dissociated hydrogen ions. Winslow and Lochridge (1906) have shown that .005 normal solutions of hydrochloric acid or sulphuric acid are fatal to typhoid bacilli in tap water, in ten minutes. This would correspond to about 250 parts of acid per million parts of sewage, at a cost of nearly a hundred dollars per million gallons. Larger quantities, too, would be necessary with sewage than with tap water on account of the organic matter present, and on account of the initial alkalinity of domestic sewage itself. Acid disinfection does not seem, therefore, to be a practicable process.

4. *Salts of Heavy Metals.* The salts of the heavy metals are stronger disinfectants than the acids. In their case, too, how-

ever, the presence of organic matter exerts an inhibition and makes the use of large quantities necessary. Metallic salts are generally costly in themselves, so that their use is rarely feasible with a substance of high organic content like sewage. Copper is the particular metal which has been suggested in this connection, from the success with which copper sulphate has been used in freeing water supplies from Algæ; but copper is relatively stronger as an algicide than as a bactericide. Johnson and Copeland (1905), in experiments at Columbus, obtained the results tabulated below; and according to these estimates a good bacterial reduction could be attained at a cost of $5 to $10 per million gallons. Other methods discussed later on, are as efficient at a less cost.

TABLE CII

DISINFECTION OF SEWAGE EFFLUENTS BY COPPER SULPHATE

(Johnson and Copeland, 1905.)

Series.	Copper sulphate, parts per million.	Per cent reduction.	
		Three hours.	Twenty-four hours.
I	5	90.0	99.90
	10	98.0	99.95
	20	98.5	99.96
II	10	40.0	99.70
	20	60.0	99.90
	40	88.0	99.95

5. *Oxidizing Agents.* The fifth class of disinfectants includes the oxidizing agents, of which three have been practically used in water and sewage work, — ozone, permanganates or manganates, and compounds yielding chlorine.

Ozone plants have been installed with more or less success for the sterilization of water at various small towns in Europe. There is no doubt as to the efficiency of the process with water, but the cost has generally been high and the apparatus subject to serious and expensive breakdowns. Methods of preparing ozone are being constantly improved and cheapened; but it can scarcely be considered a practical working process for

sewage treatment. The organic matter present would use up large amounts of the gas, and even if the supply were ample its slight solubility makes it doubtful whether solid masses of suspended matter would be adequately penetrated.

Permanganates and manganates yield oxygen both in acid and alkaline solutions, and the oxygen being in the nascent or potential state, these substances act as germicides. This germicidal action, however, is much less marked than that of either ozone or chlorine, and when used in sewage treatment it has been rather for the destruction of organic matter than for sterilization. At one time the sewage of London was treated with chlorine to destroy the obnoxious odors, but Dibdin, as chemist of the London County Council, substituted potassium permanganate for chlorine, for the express purpose of supplying oxygen, without producing sterilization, as he found that the sewage which had been partially sterilized with chlorine underwent a secondary putrefaction of a particularly offensive nature, due probably to partial destruction of nitrifying organisms.

Where disinfection is specifically aimed at, chlorine in the form of hypochlorites seems by far the most effective of all the oxidizing agents. It is the only one which at the present time is being practically used for the sterilization of sewage effluents.

The Use of Chlorine for the Disinfection of Sewage. Chloride of lime, or bleaching powder, has long been used as a deodorant and disinfectant for the direct treatment of excreta, and has been shown to be on the whole the most satisfactory material for that purpose; but its application to sewage is of recent date. A number of years ago, in certain methods of sewage treatment, as the Webster process, the Hermite process, and the Woolf process, hypochlorites were a more or less essential feature. A plant of this type was installed at Brewster, N. Y., in 1875. None of these processes were commercially successful, however, and until two years ago it was generally considered by sanitary engineers that though chlorine disinfection might be useful for special emergencies, it was far too costly for a routine method of treatment. Two important details were still imperfectly understood,

— the much greater ease with which purified effluents could be treated as compared with crude sewage, and the fact that while absolute sterilization required large amounts of chlorine a high degree of purification as regards intestinal bacteria could be attained with very much smaller quantities. The recognition of these facts brought about a revolution in the art of sewage disinfection.

The credit for the first step in this demonstration belongs to Rideal. He showed in his experiments at Guildford (Rideal, 1905) "that 30 parts of available chlorine per million would reduce the number of bacteria in crude sewage from several millions to 50,000, while 50 parts would reduce their number to 20 per c.c. Colon bacilli were reduced from one million per c.c. to less than one per c.c. by 30 parts of chlorine. In septic effluent 25 to 44 parts of chlorine per million reduced *B. coli* from two and a half to four and a half million per c.c. to less than one per c.c. With contact effluents smaller amounts of chlorine proved efficient. The primary effluent required 20 parts per million, the secondary effluent 10.6 parts per million and the tertiary effluent 2.5 parts per million to reduce the number of *B. coli* so that this organism could not be isolated in 5 c.c."

Almost immediately these results were confirmed and extended by a series of investigations carried out in this country at the Sewage Experiment Station of the Massachusetts Institute of Technology in Boston. Phelps and Carpenter (1906) found in preliminary laboratory experiments that with fairly well-purified effluents derived from more dilute American sewages even better results could be attained than those recorded in Rideal's experiments. A series of tests carried out during the summer of 1906 with the effluents from the trickling filters at the experiment station gave the results tabulated on page 384.

As a result of these tests and later ones, the conclusion was reached that trickling filter effluent could be disinfected by the addition of chloride of lime in such an amount as to yield five parts per million of available chlorine at the cost of about $1.50 per million gallons of sewage treated (Winslow and Phelps, 1907). Experiments continued at the Boston station during the last

TABLE CIII

BACTERIA IN TRICKLING FILTER EFFLUENT BEFORE AND AFTER TREATMENT WITH CHLORIDE OF LIME.

Five parts per million available chlorine. (Phelps and Carpenter, 1906.)

Date.	Bacteria per c.c.		*B. coli*, Jackson bile test.			
	Before.	After.	Before .000001 c.c.		After 1.0 c.c.	
1906						
August 11	270,000	69	+	o	+	o
13	630,000	41	o	o	+	o
14	135,000	406	+	+	+	o
15	230,000	21	o	o	o	o
16	250,000	37	+	o	o	o
18	110,000	40	o	o	+	o
20	90,000	54	+	o	o	o
21	220,000	22	o	o	o	o
23	....	....	+	o	o	o
Average	240,000	86	33%		22%	
Average removal	99.96%		999.93%			

two years have shown a continued high efficiency and have brought out a number of other interesting points. The table below, for example, shows the effect of varying quantities of chlorine and the effect of temperature upon the amounts required. Like any other chemical reaction, the poisonous action varies with the temperature in its velocity.

TABLE CIV

DISINFECTION OF TRICKLING FILTER EFFLUENT AT BOSTON. SUMMARY OF RESULTS AVERAGED BY PERIODS, TO SHOW THE EFFECT OF CHANGES IN TEMPERATURE AND IN THE AMOUNT OF AVAILABLE CHLORINE

(Phelps, 1909.)

Period.	Temp. Degrees F.	Available chlorine, parts per million.	Bacterial removal, per cent.				
			Bacteria at 20°.		Bacteria at 37°.		*B. coli*
			Total.	Liquefiers.	Total.	Acid-formers	
Nov. 12 to June 27	45	3.4	96.8	98.1	97.4	97.3	99.19
Nov. 12 to Dec. 12	42	6.3	99.6	99.7	99.8	99.9	99.99
Jan. 27 to March 28	36	3.2	95.8	97.7	96.6	96.4	98.50
April 27 to June 27	60	2.9	97.1	98.0	97.6	97.9	99.07

Meanwhile, another important series of investigations was carried out in the state of Ohio with generally similar results.

K. F. Kellerman, representing the United States Department of Agriculture, and R. W. Pratt and A. E. Kimberly, representing the Ohio State Board of Health, studied the action of chlorine and of copper on effluents of various sorts at a number of different plants in the state. They came to the conclusion that "both calcium hypochlorite and copper sulphate have high germicidal values when acting upon partially purified sewage. Calcium hypochlorite is much more rapid in its action, is more nearly able to bring about complete disinfection at a lower cost, and is less influenced by temperature and by the presence of carbonates. It is, however, liable to deterioration upon standing and is more disagreeable and less convenient to handle than copper sulphate" (Kellerman, Pratt and Kimberly, 1907). The more important results obtained with chlorine are averaged and tabulated below. The cost for treating sand effluent at Lancaster was estimated at \$5.78 per million gallons. At Marion the estimates ranged from \$8.83 for septic effluent to \$2.73 for contact effluent and \$2.43 for sand filter effluent. The high cost estimates are due to the proportionately greater expense of labor at small plants.

TABLE CV

DISINFECTION OF EFFLUENTS WITH CHLORIDE OF LIME AT LANCASTER AND AT MARION, OHIO

(Kellerman, Pratt and Kimberly, 1907.)

Series.	Available chlorine, parts per million.	Bacteria at 20°.		Bacteria at 37°.		Acid formers at 37°.	
		Initial.	Final.	Initial.	Final.	Initial.	Final.
A	4.0	130,000	140	14,000	49	840	0
B	2.8	60,000	1,600	12,000	120	3,000	0
C	4.1	225,000	1,600	120,000	390	16,000	1
D	6.0	2,000,000	700,000	900,000	230,000	70,000	24,000

Series A. Effluent of sand filter at Lancaster.
" B. Effluent of sand filter at Marion.
" C. Effluent of contact filter at Marion.
" D. Effluent of septic tank at Marion.

New Jersey and Baltimore Investigations. Along the coast of New Jersey are a number of small sewage-disposal plants, and more are being constructed every year. Shellfish layings are numerous along this coast, and the elimination of disease bacteria

is in many cases the most important part of the process of purification. Suitable areas for sand filtration are rarely available, and septic tanks and contact beds are commonly in use. As soon as it appeared that bacterial purification could be attained at a reasonable cost by chemical treatment, Professor Phelps was called in consultation by the New Jersey State Sewerage Commission; and in the fall of 1906 experiments on a practical scale were begun at the town of Red Bank.

Red Bank has a population of 6500 or more, and the average dry-weather flow of sewage is about 265,000 gallons per day. The sewage passes through grit chambers and a circular septic tank, holding about eight hours' flow. After treatment in the tank the effluent was originally passed through strainers. The use of the water below for bathing and the proximity of shellfish beds made it imperative to secure better bacterial purification. Professor Phelps, therefore, advised that a series of experiments should be made to determine the practicability of treating the effluent with chloride of lime. These were carried out during 1906 and 1907 by Mr. F. E. Daniels, acting under the direction of Professor Phelps. The general results for 1907 are indicated below.

TABLE CVI

DISINFECTION OF SEPTIC SEWAGE WITH CHLORIDE OF LIME AT RED BANK, NEW JERSEY

Bacteria per c.c. Weekly averages. (Phelps, 1909.)

Week ending.	Available chlorine, parts per million.	Total bacteria at 20°.			*B. coli* (Bile).		
		Initial.	.75 hour.	1.5 hours.	Initial.	.75 hour.	1.5 hours.
July 20	9.9	800,000	410	460	46,000	4	4
27	10.6	650,000	800	420	80,000	13	11
August 3	11.5	1,800,000	550	130	40,000	21	5
10	11.4	850,000	240	140	55,000	14	2
17	13.0	760,000	2,100	1,500	70,000	30	28
24	7.3	700,000	45,000	55,000	70,000	700	600
31	7.5	1,200,000	45,000	26,000	220,000	16,000	2,000
September 14	11.8	750,000	13,000	8,000	300,000	150	140
21	13.1	750,000	850	800	500,000	270	260
28	10.5	700,000	120	88	550,000	80	28
Average *	11.5	900,000	2,300	1,400	210,000	75	60

* Exclusive of period, August 19–31. Temperature, 56° to 58° F. throughout.

The particular interest in these experiments lies in the fact that they were conducted with septic tank effluent. The amount of chlorine required for disinfection varies directly with the amount of oxidizable organic matter present; the Ohio experiments, cited above, show that contact effluents require more chlorine than sand effluents and that septic tank liquid requires more chlorine than contact effluent. It was therefore necessary to use large amounts of chlorine at Red Bank, about twice as much as was sufficient in the Boston experiments. The table above shows that seven parts of chlorine would not do, but that ten or eleven parts per million gave good results. The State Sewerage Commission finally recommended the application of 12–15 parts per million of available chlorine, estimated to cost $3.75 per million gallons of sewage treated (Phelps, 1908). The Sewerage Commission, and later the State Board of Health, of New Jersey, have welcomed the process of chlorine disinfection for the solution of one of the pressing sanitary problems of the state, and plants of this type are now under consideration, or have been already ordered at Red Bank, Stone Harbor, Rahway and Atlantic City.

A little farther down the Atlantic seaboard is the city of Baltimore, which is just now facing a very similar condition of affairs on a much larger scale. Baltimore has a population of a little over half a million and has recently begun the construction of a comprehensive system of sewerage to remove the present pollution of the Patapsco River. As soon as it was proposed to discharge all the sewage of the city at a single point the oyster industry of Chesapeake Bay was aroused, and in calling upon experts for advice as to disposal, the Sewerage Commission specified that " the effluent proposed to be discharged into Chesapeake Bay or its tributaries in the system to be recommended by the engineers shall be of the highest degree of purity."

The Board of Advisory Engineers recommended trickling filters as a primary treatment for the sewage, and in its first report (Baltimore, 1906) suggested supplementary sand filters for accomplishing bacterial purification. The cost of the supplemen-

tary treatment alone was estimated at over a million dollars for construction and $55,000 a year for operation. When the disinfection results obtained at the Technology experiment station were published, Professor Phelps was invited to conduct experiments at Baltimore, and did so during the summer of 1908 in coöperation with Mr. E. B. Whitman, engineer in charge. The general results of these tests have recently been published in an exhaustive review of the whole subject, as a bulletin of the United States Geological Survey (Phelps, 1909).

The chlorine was applied, in the Baltimore experiments, to the effluent of one of the experimental trickling filters at the Walbrook Testing Station, a twelve-foot bed of half-inch to one-inch stone. The bleaching powder was kept down to an amount which would supply, on an average, about two parts per million of available chlorine. The results were good, considering the small amount of disinfectant used; the average reduction of bacteria determined on gelatine at 20° was 95.8 per cent; of total bacteria at 37°, 94.9 per cent; of acid formers at 37°, 97 per cent; of *B. coli* as determined by the lactose bile test, 90 per cent. These results would seem to indicate the advisability of increasing the dose of chlorine to something nearer the five parts per million found necessary at Boston. On the whole, however, these experiments were considered successful. The Board of Advisory Engineers has recommended the substitution of chlorine treatment for the supplementary sand treatment originally planned. The cost of operation will be somewhat less than that of the sand filters; and the capital cost will be merely nominal. The net saving to the city by the adoption of chlorine disinfection should be in the neighborhood of a million dollars.

German Experiments on Chlorine Disinfection. In Germany experimental results have been somewhat less favorable to the chlorine treatment, and the general conclusion has been that disinfection was only suitable for special emergencies and perhaps for the treatment of hospital sewage. Recent observers (Schumacher, 1905; Kranepuhl, 1907; Kurpjuweit, 1907) report that large amounts of chlorine are necessary, in the neighborhood of

60 parts per million parts of sewage. This is partly because these investigators insist on an extreme degree of purification, the criterion of success being the absence of *B. coli* in a large proportion of cases from one litre samples. As pointed out by Rideal, absolute sterilization requires very large doses of chlorine; while much smaller amounts will effect a 99 per cent reduction. The table below, from results obtained by Dunbar, indicates that a fair per cent reduction can be effected by 33 to 50 parts per million of chloride of lime when crude German sewage is disinfected. Estimating that the chloride of lime contains about one-third of available chlorine and remembering that crude sewage was treated, these results seem to promise success in the disinfection of effluents, even with a sewage as strong as that of most German cities. Dunbar himself notes that septic tank effluent required only one-fifth as much chlorine for disinfection as crude sewage. This would bring the amount necessary for 99 per cent disinfection of septic effluent down to 10 parts per million of available chlorine, about the same as that indicated by experiments in America.

TABLE CVII

DISINFECTANT ACTION OF VARIOUS AMOUNTS OF CHLORIDE OF LIME

Crude German Sewage. (Dunbar, 1907.)

Chloride of lime,* parts per million.	Bacteria per c.c. Before.	After.
500	1,350,000	15
200	1,350,000	23
100	1,350,000	36
50	1,350,000	72
33	1,350,000	3,620
25	1,350,000	59,000

* Not available chlorine, as in other tables.

Supplementary Applications of the Disinfection Process. The primary object of chlorine treatment is of course bacterial removal and not chemical purification. In connection with other properly designed works for the purification of sewage, chlorine may, however, also be used for a certain amount of direct chemical oxidation. Experiments by Rideal in 1904–05 proved

that bleaching powder when used in small quantities is sufficient to destroy foul odors, such as those of a septic effluent, and at the same time acts as a powerful oxidizer and decreases the work required of the filters. The results of a long series of experiments, from August, 1906, to August, 1907, carried out at Guildford, are summarized as follows (R. S. C., 1908):

1. Treatment with a hypochlorite solution containing available chlorine equal to 35-50 per cent of the amount of oxygen consumed by permanganate in 5 minutes in the cold is sufficient to do away with the smell of hydrogen sulphide, leaving only an inoffensive odor of spent bleach and fresh sewage.

2. The addition of this quantity in no way interferes with the efficiency of the filter, but adds to it by helping to keep down an excess of gray growths on the top.

3. A much larger quantity may be added without any danger to the filter, somewhere between 200 and 500 per cent being the limit of safety.

4. A dose of oxychloride more than sufficient to remove the hydrogen sulphide smell and kill *B. coli* in the liquid may be used without prejudice to the purifying ability of a mature percolating filter.

The author draws these further conclusions: (1) that sewage from hospitals may be freed from dangerous organisms by the use of oxychloride before passing into beds or entering ordinary sewerage systems without interfering with the usual methods of purification; (2) that when beds are clogged with growths these can be dissolved and washed through speedily by occasional doses of oxychloride. From a third series of similar experiments with each filter divided into two compartments, one containing a fine and the other a coarse filtering medium these conclusions are drawn: (1) that a bed can be successfully matured when using oxychloride in such quantities as are required for preventing offensive odors; (2) that this treatment renders possible the use of fine grade filters; (3) that it renders easy the cleaning of pipes, sprinklers, and siphons blocked by growths, without disconnecting the system.

The Design of a Disinfecting Plant. The production of chlorine for disinfecting purposes by electrolytic methods is still, as always, theoretically attractive. At the present time, however, bleaching powder containing 35 per cent available chlorine is made very cheaply by the action of chlorine obtained by electrolysis of a brine solution on slaked lime, and it can be purchased (1909) in any large city in the Eastern United States for about twenty-five dollars a ton. Under these circumstances it has not yet been demonstrated that any electrolytic process for making chlorine at the sewage plant, unless free power be available, is as economical as the purchase of the ordinary commercial product. The disinfection of sewage, therefore, resolves itself into the simple problem of adding a solution of bleaching powder to the sewage and storing it for a short period of time.

A plant for disinfecting a sewage effluent should consist of three tanks, — a solution tank, a dosing tank and a sterilizing tank. The solution tank is used for the preparation of a saturated solution of the disinfectant made by adding bleaching powder to water and for retaining the sludge that is formed. The size of the tank depends upon the amount of sewage to be treated; one of 300 gallons' capacity would be sufficient for the treatment of a million gallons when the bleaching powder required did not amount to more than 15 parts per million. From the solution tank the liquid runs to the dosing tank, from which it is added to the sewage by means of gauged outlets, so that the amount added can easily be adjusted. These outlets may be connected with pipes arranged in the form of a grid, so that the disinfecting liquid comes in contact with the sewage at various points, and to bring about a thorough mixture it is well to run the sewage through a channel containing baffling boards.

The disinfection tanks, into which the sewage then passes, should be of sufficient size to contain approximately one hour's flow.

The action of the disinfectant is very rapid at first, and is then much slower, as the chlorine is used up by combination with the

organic matter present. After a time, indeed, the total number of bacteria in the disinfected effluent begins to rise again as the chlorine disappears and renewed multiplication sets in. This secondary increase does not, however, of course include the pathogenic forms. The results, tabulated below by Phelps (1909), from some of the Technology experiments, indicate that most of the disinfectant action takes place in the first ten minutes, and that a storage of one hour is ample for all practical purposes. It must be remembered that chlorine exerts a vigorous corrosive action upon metals; and this must be guarded against in the construction of this part of the plant. The tanks themselves may be of wood or masonry. The pipes and connections should be of phosphor-bronze or cast iron; and all parts should be so arranged that they can readily be taken apart for cleaning.

TABLE CVIII

RELATION BETWEEN TIME OF CONTACT AND EFFICIENCY OF DISINFECTION WITH CHLORIDE OF LIME

Available chlorine, 5 parts per million.
Boston experiments. (Phelps, 1909.)

	Bacteria Remaining per 1,000,000 initial.			
Time of contact.	10 min.	15 min.	1 hour.	2 hours.
August 6	....	1,100	160	150
9	2,500	190	58	7
10	10,000	270	....	40
11	3,500	570	154	100
14	47,000	1,100	700	570
15	4,200	210	120	120
16	1,200	240	160	130
17	9,800	800	260	150
20	400,000	12,000	7,000	5,500
21	28,000	2,100	1,300	1,000
23	1,300	230	110	31
Average	50,000	1,700	950	700
Per cent	5.0	0.17	0.10	0.07

Fig. 112, from a design prepared by Prof. E. B. Phelps for Rahway, N. J., gives a good idea of a disinfecting plant for a small town.

The Cost of Disinfection. The main element in the cost of disinfection is the amount of bleaching powder required, though of course the cost of the tanks, maintenance and labor must all be taken into account.

The price of the bleaching powder will depend on the nearness of the source of supply, and the cost of tanks will depend somewhat on the organic purity of the liquid. With well purified effluents, requiring less than five parts per million of available chlorine, the storage may with advantage be prolonged over one hour, as the chlorine will not disappear as quickly as with effluents of higher organic content. The cost of labor will depend on local conditions, and for a small plant will be proportionately higher, though in a small community the

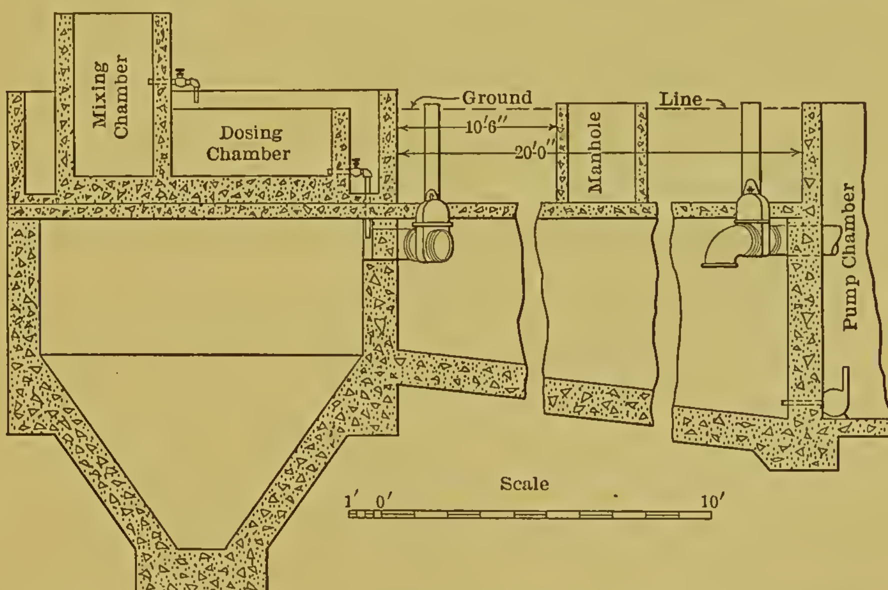


FIG. 112. Detritus Tank and Disinfecting Tank at Rahway, N. J. (courtesy of E. B. Phelps).

care of the plant may be combined with other town work. The cost of treatment with varying amounts of bleaching powder, from one part to fifteen parts of available chlorine, has been estimated by Professor Phelps (Phelps, 1909); and according to his figures the application of five parts of chlorine, required for the effluent of a trickling filter, will cost in the neighborhood of \$1.75 per million gallons. To disinfect crude sewage with fifteen parts per million of chlorine will cost about \$5.00 per million gallons.

CHAPTER XIII

ANALYSIS OF SEWAGE AND SEWAGE EFFLUENTS

General Features of Sewage Analysis. The strength and general character of a sewage, or of a sewage effluent, depends on the amount and nature of the dissolved and suspended solid matter it contains and on its bacterial content.

The efficiency of any process of sewage treatment depends not only on the difference in the amount of mineral and organic matter before and after treatment, but on the changes that the organic matter has undergone and on the diminution in the number of bacteria that has taken place.

A complete chemical analysis of a sewage or a sewage effluent would include the following determinations:

Total solids	Total, Fixed, Volatile.
Suspended solids	Total, Fixed, Volatile.
Dissolved solids	Total. Fixed, Volatile.
Nitrogen as	Free ammonia, Total albuminoid ammonia, Albuminoid ammonia in filtered solution, Total organic nitrogen, Organic nitrogen in filtered solution, Nitrites, Nitrates.
Total oxygen consumed and oxygen consumed in filtered solution	3 minutes at 80° F. 4 hours at 80° F. or 2–10 minutes at 212° F.

Oxygen dissolved.
Combined chlorine.
Fats.
Acidity or alkalinity
in terms of sulphuric acid
or sodium hydroxide.

To determine the strength and general character of sewage and sewage effluents, and the efficiency of any process of sewage treatment, chemical, and in certain cases bacterial, examinations are required.

The bacterial examination should include the determination of the total number of bacteria present in a given volume of the liquid and the approximate number of colon bacilli; and in special cases it may be desirable to extend the analysis so that knowledge regarding the pathogenic bacteria present can be obtained.

The primary object of sewage purification is the elimination of offensive decompositions. The study of putrescible organic matter by the nitrogen and oxygen consumed determinations and by special putrescibility tests constitutes, therefore, the most important problem in sewage analysis. Estimations of suspended solids are also of great importance, since the deposition of decomposable matter may often cause a nuisance, even when the original effluent as a whole is stable; and in studying processes of preliminary treatment this determination is of prime importance. Bacterial examinations are in general required only in special cases where shellfish beds or water supplies are directly menaced.

Sewage Sampling. The value of the data obtained from the analyses of sewage and sewage effluents depends upon the method employed for obtaining the sample for analysis. The sample to be analyzed should be a representative one, and should contain a true average amount of the various substances occurring in the sewage or sewage effluent.

Sewage and sewage effluents vary from hour to hour, both in volume and quality, and the analysis of a casual sample, or a number of casual samples, gives no trustworthy information. Day samples are often 25–50 per cent stronger than the average for the twenty-four hours. Figure 113 indicates the actual course of the daily variations in Boston sewage (Winslow and Phelps, 1905). The chlorine curve in this case is influenced chiefly by the tides, since a considerable proportion of sea water backs up into the intercepting sewers through leaking tide gates;

and the effect of a slight shower is manifest between seven and eight in the evening; but the other data indicate a fairly normal relation for the sewage of a large city in dry weather.

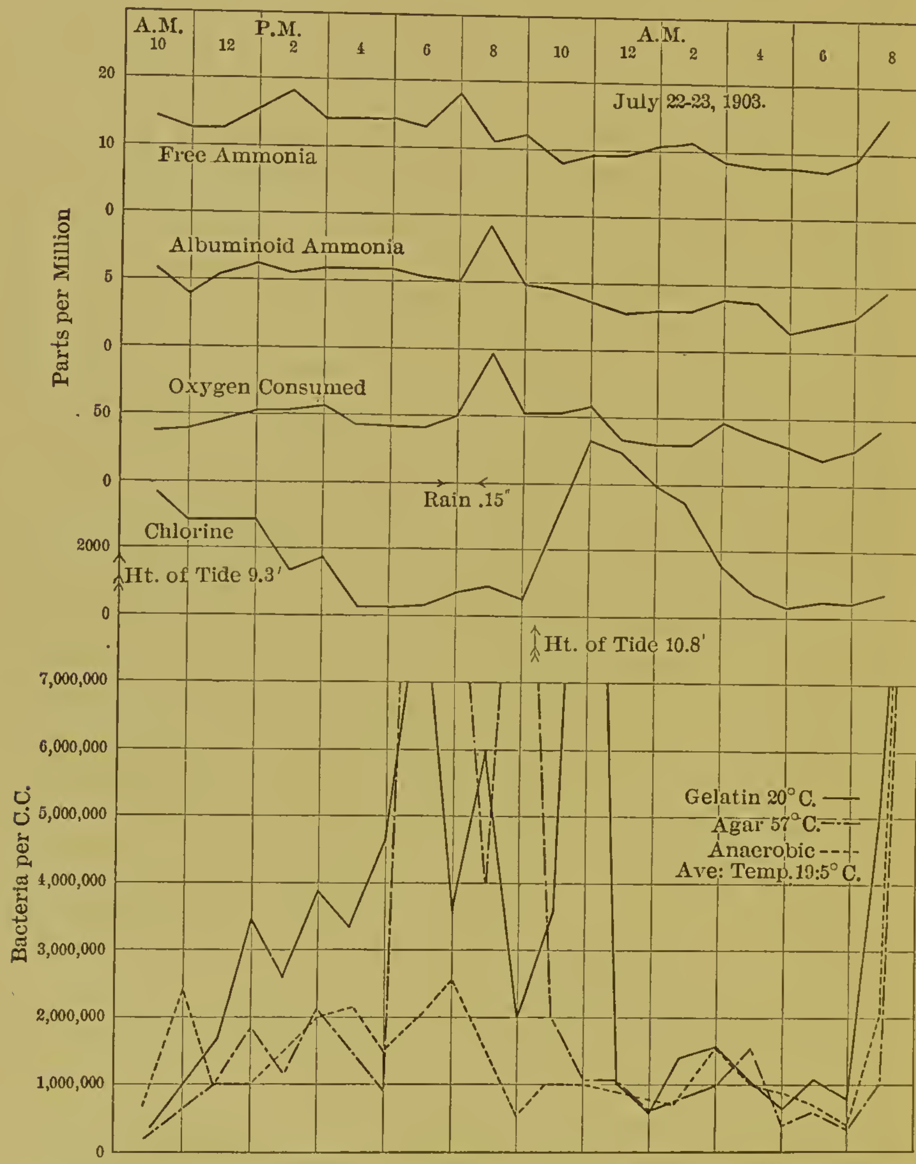


FIG. 113. Diagram of Hourly Variations in Composition of Boston Sewage.

To obtain a true representative sample, hourly or half-hourly samples must be taken throughout a given period and mixed together in proportion to the flow.

At sewage experiment stations and at certain large plants the hourly or half-hourly collection of samples constitutes a part of the regular routine work, but at a majority of sewage plants this is not done on account of the labor required; and the strength of the sewage and sewage effluent is determined from analyses of samples collected during a comparatively short period of time. When this is the method, the collection of samples should be made in dry weather and extended through seven days, though in case of small works the period may possibly be reduced to two or three days, which should not include Saturday, Sunday or Monday. The rainfall for the preceding seven days and during the period of collection should be noted. In order to avoid multiplication of analyses, each half-hourly or hourly sample is placed in a one or two liter glass-stoppered bottle on which is noted the rate of flow and time of collection, and at the end of each twelve or twenty-four hours, a composite sample is made, the amount of each separate sample taken being determined by the rate of flow noted on the bottle. The separate samples before being mixed should be placed, if possible, in an ice box, and if the analysis of the composite sample is delayed and no bacterial determinations are to be made, it is advisable, in order to prevent change in the organic matter, to sterilize the sample by the addition of chloroform, five to ten cubic centimeters per liter of sewage.

The method adopted at the Columbus Experiment Station as given by Johnson (1905) is an excellent illustration of the proper method of obtaining a representative sample where the flow is constant:

"The samples of the crude sewage for chemical analysis were collected from the screen chamber after screening, as before stated, at half-hourly intervals throughout the twenty-four hours. These half-hourly portions were collected in four ounce bottles, which were completely filled with sewage, tightly stoppered and placed at once in an ice chest at a temperature of about 10 degrees C. As a midnight to midnight period was adopted as the station's official day, the first portion of crude sewage for the day was collected at 12.30 A.M., and the last and 48th portion at 12.00 P.M. At 8.00 A.M. on the day following, or 32 hours after the collection

of the first portion, the contents of the 48 small bottles were mixed in equal proportion in a large glass bottle and the chemical analysis of the composite sample was immediately begun. . . .

"Samples for bacterial analysis were collected in duplicate in sterilized wide-mouthed glass-stoppered bottles at the same points as were those for chemical analysis. On account of the material changes in the bacterial content of the sewage which would take place in the bottles when stored, it was found to be inadvisable to store the bacterial samples for periods of more than about one hour. Except for occasional series of hourly samples, collected in order to ascertain the variations in the bacterial content of the sewage throughout the twenty-four hours of the day, all bacterial samples of the crude sewage were collected at 7.00 A.M., 11.00 A.M. and 5.00 P.M. These times roughly covered the weakest, strongest and medium strength sewage of the day and enabled representative average results to be obtained. All bacterial samples were plated as soon as collected."

The methods of sampling so far described apply particularly to cases where the flow is continuous and where the efficiency of the whole plant is in question. To test the efficiency of certain portions of the process where the flow can be regulated, as, for instance, the working of a contact bed or an intermittent filtration bed which is receiving clarified sewage, very fair representative samples can be obtained by mixing equal proportions of three or four samples taken of the liquid as it enters the bed and of the liquid as it leaves the bed. In order that the samples taken at the inlet and outlet should be comparable, it is evident that account must be taken of the time the liquid remains in the bed.

Chemical Determinations. The determination of the total suspended matter and the proportion that is volatile at a red heat is included in almost all sewage analyses, as upon the data thus obtained the efficiency of the preliminary process of sewage treatment can best be judged, and also because, as has been stated already, the putrescibility of a sewage effluent may often depend on the suspended matter it contains. The turbidity of sewage has little significance, as it depends not only on the amount of suspended matter, but also on the size of the floating particles. The suspended matter can best be determined directly

by filtration through a Gooch crucible or indirectly by filtering through paper.

In regard to the other constituents, authorities in America generally consider that more valuable information can be obtained from the study of the nitrogen data than from any of the other factors, while in England and Germany greater importance is placed on the oxygen consumed. This factor has been used in England as a very rough guide for the classification of sewage, "strong sewage" being considered as one which absorbs from a very strong solution of potassium permanganate in four hours at 80° F. 170–250 parts of oxygen per million, "average sewage" one which absorbs 110–120 parts, and "weak sewage" 70–80 parts.

Though the nitrogen factors and the oxygen consumed probably give more information than any of the other factors, the analysis of sewage is seldom if ever restricted to these determinations, and the general custom in America at the present time is to include in all such analyses the determination of total solids, suspended solids, ammoniacal nitrogen, organic nitrogen before and after filtration, nitrogen as nitrites and nitrates, oxygen consumed before and after filtration, and chlorine.

Formerly the nitrogen as albuminoid ammonia was always determined, and many authorities believe that it is a mistake to have substituted the determination of organic nitrogen for nitrogen as albuminoid ammonia, and consider that if the nitrogen in both forms is not to be determined, the nitrogen as albuminoid ammonia should be retained as giving, on the whole, more valuable information than the organic nitrogen. The determination of organic nitrogen does furnish data from which a fair approximation of the total nitrogenous matter in the sewage can be made, but it gives no idea as to the amount of nitrogenous matter which is in a stable condition, like the so-called "humus," in distinction from the nitrogenous matter that is easily broken down, like certain of the proteids and amido acids. On the other hand, the nitrogen as albuminoid ammonia, though giving little information regarding the total amount of

nitrogenous matter, does give an idea as to the amount of nitrogenous matter that is most easily decomposed, and on account of the great number of analyses of sewage where this factor has been determined, it serves also as a very useful guide in the classification of sewage.

The nitrogen as free ammonia is useful in giving an idea as to the amount of nitrogenous matter that has been decomposed, and in the analysis of effluents yields information as to the efficiency of the process of treatment, since nitrogen as free ammonia indicates incomplete oxidation. Nitrogen as nitrites usually indicates instability of an effluent, but nitrogen as nitrates implies stability and efficiency of treatment.

The oxygen consumed does not give the total amount of oxygen necessary to oxidize completely all the organic matter present, for which no accurate method of determination is known, but only the amount given up by potassium permanganate to sewage or effluent under certain conditions. This information is valuable in determining the strength of a sewage and is of assistance in deciding upon the degree of purification effected by a given process.

In America the oxygen consumed is determined by adding sulphuric acid and a solution of potassium permanganate to the liquid and heating to 212° F. for two, five or ten minutes. In England, the temperature at which the liquid is allowed to stand is usually 80° F., and the results are observed after three minutes and again after four hours.

The data obtained by these various methods of procedure are very variable and give rise to much confusion in making deductions from results accumulated at various places. Attempts have been made to obtain factors which could be used for converting to an equivalent basis the results obtained by the different methods, and the following table prepared by Fuller (1903) based on results given in Blair's "Organic Analyses of Waters," and on data supplied by Kinnicutt, is of some assistance in making comparisons, though it should be considered only as a rough approximation.

TABLE CIX

APPROXIMATE COMPARISON OF AVERAGE AMOUNTS OF OXYGEN CONSUMED BY SEWAGE AND SEWAGE EFFLUENTS AS SHOWN BY DIFFERENT METHODS

Method.	Temperature of solution.	Period of contact.	Relative results.
Kübel, as practised at Boston and generally in America	Boiling	5 minutes	1.00
Kübel, as practised at Lawrence, Mass.	Boiling	2 minutes	0.65
Kübel, as practised in Germany *	Boiling	10 minutes	1.25
	80° F.	3 minutes	0.20
English official tests	80° F.	15 minutes	0.35
	80° F.	4 hours	0.60

* German results generally refer to "permanganate consumed" and should be divided by four to give oxygen consumed.

The English methods of determining oxygen consumed in three minutes and four hours throw light on the ratio between the more easily oxidizable substances, such as hydrogen sulphide, nitrites, urea, ferrous salts, etc., and those compounds which are not so easily acted upon by oxygen; and consequently they give more information than the American procedure. There is a tendency, however, at the present time, in the United States, to adopt the English method of making two determinations, but changing the four hours at 80° F. to ten minutes at 212° F.

The determination of chlorine or sodium chloride is of considerable importance in the analysis of domestic and residential sewage, as the chlorine in this case is chiefly derived from excreta and household waste. In sewage containing any large amount of trade waste it has only a local value as indicating whether or not the samples of sewage and effluent taken for analysis correspond.

The opinion of English authorities as to the determinations which are most essential is shown by the following statement taken from the Sewage Report of the Local Government Board (Moore and Silcock, 1909):

"The analysis should in all cases include the following items: (1) Ammoniacal nitrogen. (2) Albuminoid nitrogen. (3) Total nitrogen. (4) Oxygen absorbed from strong permanganate in three minutes at 80 degrees F. (5) Oxygen absorbed from

strong permanganate in four hours at 80 degrees F. (6) Suspended solids. (7) Soluble solids. (8) Chlorine.

"It is also desirable that the amount of dissolved oxygen taken up during the oxidation of the ammoniacal and organic matter of the sewage should be given."

To the above list the determination of the amount of fat a sewage contains might well be added, for the procedure is not difficult, and the information thereby obtained may be of importance in the consideration of methods of treatment, and, when studied in connection with the nitrogen data and oxygen consumed, gives some indication of the amount of celluloses present in the sewage.

Standards of Purity. Various standards of purity have from time to time been formulated in England, and the different views held as to when an effluent should be deemed polluting and inadmissible into a stream are shown by the provisional standards of purity of certain well-known commissions. The Mersey and Irwell Joint Board requires that the effluent shall not contain, per million parts, more than 1.4 parts albuminoid ammonia, 14.3 parts oxygen absorbed in 4 hours at 60° F., 3.6 parts oxygen absorbed in 3 minutes at 60° F., and 43 parts solids in suspension, and that it shall be nonputrescible by incubator test for 3 to 5 days at 75° F. The Ribble Joint Committee consider an effluent as unsatisfactory if the albuminoid ammonia ranges between 1.5 and 2 parts per million, and bad if the albuminoid ammonia is over 2 parts, and require that the suspended matter shall not exceed 30 parts per million. The Thames Conservancy Board deem an effluent containing 30 parts of organic carbon and 11 parts of organic nitrogen per million unsatisfactory.

The desirability of standards of purity to which all sewage effluents should be made to conform has always aroused much controversy, the inherent difficulty being that no account is taken in such standards of the nature or of the volume of the water, or the use that is made of the water, into which the effluent is emptied. Furthermore, the effluent of a trickling filter contain-

ing an amount of organic matter considerably in excess of the above standards may often be stable on account of the simultaneous presence of a large amount of dissolved oxygen. Consequently such standards have not been generally recognized in America. In the United States the usual requirement is that the effluent must be nonputrescible, or must contain only that amount of organic matter which, when the effluent is emptied into a stream, can be oxidized by the oxygen contained in the water at the time of minimum flow. In certain cases, as when the effluent is emptied into a stream in the near neighborhood of a point at which water is taken for domestic use, or is emptied into salt water in the neighborhood of shellfish layings, the effluent must be free from pathogenic bacteria.

The most important points, therefore, to be determined regarding a sewage effluent are whether or not the effluent is putrescible, whether it will improve or deteriorate when emptied into a given water-course, and whether or not it contains intestinal or typical sewage bacteria.

As a general statement a sewage effluent is not putrescible when it contains sufficient oxygen, either as free oxygen or combined oxygen in the form of nitrates, for the complete oxidation of the nitrogenous organic matter and the sulphur compounds contained in the effluent, though a complete statement should also take into account the carbohydrates and fats.

The Manchester Test for Putrescibility. Up to the present time no test has been devised which directly measures the amount of putrescible matter in sewage or effluents, and we are therefore obliged to content ourselves with the information given by indirect methods. Among the earliest of these indirect methods is the so-called Manchester test. This test consists in determining the amount of oxygen absorbed from potassium permanganate in three minutes at room temperature, and the amount that is absorbed in three minutes at room temperature after the sample has been kept in a tightly closed bottle in the incubator at 98 degrees F. for seven days. By incubation the organic compounds which may still be contained in the effluent

are broken down by the bacteria into simpler products, and if the effluent contains sufficient oxygen to be nonputrescible these simpler compounds are completely oxidized, while if it does not contain sufficient oxygen they remain more or less unchanged, and absorb oxygen very quickly when potassium permanganate solution is added to the incubated effluent. Consequently, if the amount of oxygen absorbed from potassium permanganate in three minutes after incubation is greater in amount than that absorbed before incubation, the effluent is considered deficient in oxygen and consequently putrescible.

According to Stoddart (Stoddart, 1901), since the formation of hydrogen sulphide runs parallel with, and is no doubt the chief cause of, the increase in the oxygen consumed, the test may be simplified by substituting lead chloride for the permanganate process, and if necessary a calorimetric test may be made. Stoddart considers, however, that a visible darkening of the incubated solution on addition of the lead salt is all that is required, and suggests this test for official purposes.

The Manchester method for determining the putrescibility of effluents is considered by many authorities as unreliable, and there is little question that certain effluents which remain sweet and develop no odor take up more oxygen from potassium permanganate after, than before, incubation. On this account it is held that more satisfactory information as to putrescibility can be obtained from the determination of dissolved oxygen.

The Oxygen-Dissolved Test. This test consists in determining the amount of free oxygen the organic matter in the effluent requires for oxidation. Atmospheric oxygen is introduced into the effluent by means of aeration, or by adding to the effluent a given volume of tap water saturated with air. The amount of free oxygen the liquid then contains is determined, and the amount taken up by the organic matter is shown by the amount of free oxygen remaining in the liquid at the end of a given period. Harrison considers an effluent as satisfactory or nonputrescible if it does not absorb more than 3 c.c. of dissolved oxygen per liter, after standing at room temperature for 24

hours; Fowler if, after the addition of tap water, 1 volume of effluent to 9 volumes of tap water saturated with oxygen, and heating to 80° F. for 48 hours, it does not absorb more than 30 per cent of the total volume of dissolved oxygen present in the mixture before incubation; Stoddart if, under the same conditions but heated to 75° instead of 80°, it does not absorb more than 1.66 grains of dissolved oxygen per gallon (R. S. C., 1908, Appendix VII). This general method for determining putrescibility has the approval of the Royal Commission on Sewage Disposal, as shown by the following statement (R. S. C., 1908):

"According to our present knowledge, an effluent can best be judged by ascertaining, first, the amount of suspended solids which it contains, and, second, the rate at which the effluent, after the removal of the suspended solids, takes up oxygen from water.

"In applying this test it is important that the suspended solids should be removed, and estimated separately.

"For the guidance of local authorities we may provisionally state that an effluent would generally be satisfactory if it complied with the following conditions:

(1) That it should not contain more than 3 parts per 100,000 of suspended matter; and

(2) That, after being filtered through filter paper, it should not absorb more than:

(*a*) 0.5 part by weight per 100,000 of dissolved or atmospheric oxygen in 24 hours.

(*b*) 1.0 part by weight per 100,000 of dissolved or atmospheric oxygen in 48 hours; or

(*c*) 1.5 parts by weight per 100,000 of dissolved or atmospheric oxygen in 5 days."

The Organic Sulphur Method. Dunbar (1908) states that the sign of putrefaction is the formation of hydrogen sulphide, and that the hydrogen sulphide is formed from the sulphur present in the organic matter, — the organic sulphur; and believes, as the result of numerous experiments, that when the biological purification of sewage has been carried to the extent which is sufficient to remove all the organic sulphur, the effluent will be nonputres-

cible. To decide whether an effluent is or is not putrescible, it is therefore, according to Dunbar, only necessary to determine the presence or absence of organic sulphur. The method of determining organic sulphur, known as the Hamburger putrescibility test, consists in removing all the inorganic sulphur in the effluent as barium sulphate and then changing the organic sulphur, by heating with potassium, into potassium sulphide, which when once formed can be detected by Caro's methylene blue reaction. By this method effluents can be quickly tested, but the test cannot be considered perfectly satisfactory since it does not take into account the amount of organic sulphur the effluent contains; and though, as Dunbar believes, putrescible effluents probably always contain organic sulphur, it is still open to question whether all effluents which contain organic sulphur are putrescible. It seems as though this must depend on the ratio that exists between the organic sulphur and the oxygen, free and combined as nitrates, contained in the effluent. To determine this ratio the amount of organic sulphur must be known, and experiments are now being carried on at the Hamburg Hygienic Laboratory to improve the present test so that it will give quantitative results.

The Methylene Blue Test. To obtain results by the oxygen consumed, the oxygen dissolved, or the organic sulphur method requires not only a well-equipped laboratory, but also chemical training, and on this account these tests are not well adapted for the daily testing of effluents at many sewage disposal plants. Methods which require only simple apparatus and no special training have been proposed from time to time, the best being the test, originally devised by Spitta (1901) and improved by Spitta and Weldert (1906).

This test depends upon the fact that methylene blue, the zinc salt of tetramethylthionine chloride ($C_{16}H_{18}N_3SCl$), or methylene green, the zinc salt of mononitromethylene blue, are usually broken down into colorless derivatives by various substances formed during the decomposition of the organic matter contained in sewage, and especially by hydrogen sulphide. The

method consists in adding a small amount of the methylene blue or methylene green, dissolved in water, to the effluent, in a tightly stoppered bottle, and noting the number of days required for the color to be discharged. The time required for decolorization depends of course upon the temperature, being about twice as long at 70° F. as at 98° F. If the color is not discharged in 14-days at 70° F., the effluent may be considered as nonputrescible. Many consider an effluent which retains its color for five to six days as sufficiently stable to discharge into a stream.

Phelps (1909), as a result of numerous experiments, gives a table in which the degree of putrescibility is determined by the number of days required to discharge the color:

TABLE CX

RELATION BETWEEN REDUCING TIME AND RELATIVE STABILITY AT 70° F.

Reducing time in days.	Relative stability.	Reducing time in days.	Relative stability.
1	21	9	87
2	37	10	90
3	50	11	92
4	60	12	94
5	68	14	96
6	75	16	97
7	80	18	98
8	84	20	99

The methylene blue test, like the oxygen consumed and the organic sulphur method, has been criticised on the ground that it sometimes indicates putrescibility when the effluent is really stable. This is probably due to the fact that in certain cases the amount of decomposable organic matter remaining in the effluent after the oxygen has all been used up, even though sufficient to reduce methylene blue, is so small in amount that during its decomposition, even under anaerobic conditions, no noticeable odor is developed.

Another criticism is sometimes made to the effect that substances like iron sulphide and hydrogen sulphide bring about a

quicker discharge of color than putrescible organic compounds. There is very little force in this criticism, for hydrogen sulphide, according to most authorities, is one of the products always developed in putrescible effluents; and if effluents contain iron sulphide and hydrogen sulphide they are certainly in an unstable condition.

Another test which has often been used for determining putrescibility is known as the odor test. It consists in completely filling a bottle with the effluent, inserting a tightly fitting stopper, incubating the sample for one week at 98° F., and noting if at the end of that time a disagreeable odor is developed. This test as shown by Eddy gives, as a rule, reliable information, but the personal equation is too great for it to be recognized as an official method.

Tests of putrescibility may be applied, not only to the sewage effluent, but to a mixture, in any desired proportion of the effluent itself and the water of the stream into which it is to be discharged. The latter method is extensively used in England and gives valuable information as to the probability of a stream becoming obnoxious on the addition of the effluent, but it should not exclude direct tests on the effluent itself.

The determinations which have been mentioned in this chapter are those which are usually employed in determining the strength and character of a town or city sewage, in determining the efficiency of processes of treatment for such sewage and in determining the stability of effluents, but no mention has been made of such estimations as those of iron, calcium, magnesium, free mineral acids, etc., which, though often required when the purification of trade waste is under consideration, are of little importance in connection with general sewage disposal, and consequently are foreign to the subject-matter of this book.

In this chapter, also, no attempt has been made to describe the methods of analyses, for such an attempt would only have resulted in repeating what has been stated in books devoted to the analysis of water and sewage; and for information on this subject we can refer our American readers to the Reports of

the Committees on Standard Methods of Water and Sewage Analysis to the Laboratory Section of the American Public Health Association, published by the Chicago University Press and in the American Journal of Public Hygiene. English and German methods differ more or less from those used in America, but are not so different that it would be at all difficult for an international commission on sewage analysis to formulate a universal standard.

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